CALIFORNIA
DEPARTMENT OF TRANSPORTATION

manual change transmittal

TITLE	DIVISION OF DESIGN HIGHWAY DESIGN MANUAL SIXTH EDITION – CHANGE 08/01/11	APPROVED BY TERRY L. ABBOTT, Chief	NO. Date Issued: 08/01/11 Page 1 of 4
SUBJECT AREA	Table of Contents; List of Figures; List of Tables; Chapters: 700, 800, 810, 850, 880 and 870	ISSUING UNIT DIVISION OF DESIGN	
SUPERCEDES	SEE BELOW FOR SPECIFIC PAGE NUMBERS	DISTRIBUTION **ALL HOLDERS OF THE 6TH EDITION, HIGHWAY DESIGN MANUAL**	

The Table of Contents; List of Figures; List of Tables; Chapters 700; 800, 810, 850, 880, and 870 of the Sixth Edition, Highway Design Manual (HDM) have been revised. The update primarily relates to the Departmental conversion of AASHTO LRFD structural design standards and Standard Specifications reference revisions that are needed to conform to the wording in the 2010 Edition of the Standard Specifications released late May 2011. The changes are described in the summary below with change-sheets available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm. These revisions and changes are effective August 1, 2011, and shall be applied to on-going projects in accordance with HDM Index 82.5 – Effective Date for Implementing Revisions to Design Standards.

HDM Holders are encouraged to use the most recent version of the HDM available on-line at the above website. Should a HDM Holder choose to maintain a paper copy, the Holder is responsible for keeping their paper copy up to date and current. Using the latest version available on-line will ensure proper reference to the latest design standards and guidance. If you would like to be notified automatically of any significant changes or updates to the HDM, go to http://www.dot.ca.gov/hq/oppd/hdm/hdmlist.htm.

A summary of the most significant revisions are as follows:

Index 705.1(a) — **Special Treatments and Materials, Page 700-4**
Added green vinyl-clad mesh among the choices for fence treatments.

Index 807.2 — **Federal Highway Administration Hydraulic Publications, Page 800-36**
Update references to currently available FHWA hydraulic publications.

Table 808.1 — **Summary of Related Computer Programs and Web Applications, Page 800-39**
Revise Table of recommended hydraulic software programs and web applications to include new NOAA Atlas 14 and USGS Streamstats web tools as well as Departmental acquisition of new RDS software from Autodesk.

Index 813.8 — **Debris, Page 810-5**
The reference where guidance on special considerations are given to highways located across desert washes was corrected.

Index 815.3(2) — **Federal Agencies, Page 810-7**
Reference to the National Water Data Exchange as a source for water related data has been replaced with the USGS Office of Surface Water website.

Index 815.3(3)	**State Agencies, Page 810-8** Eliminate explanation of how the Department of Water Resources became the primary state agency collecting stream-gaging and precipitation data.
Index 819.2	**Empirical Methods, Page 810-15** Reference for recommendations regarding intensity-duration-frequency IDF curve generating software was corrected.
Index 819.5	**Transfer of Data, Page 810-21** Eliminate reference to old hydrology tool, intensity-duration-frequency IDF-32, and emphasize recommendation for use of new NOAA Atlas 14 IDF tool.
Table 819.5A	**Summary of Methods for Estimating Design Discharge, Page 810-22** Eliminate reference to old hydrology tool, intensity-duration-frequency IDF-32, and emphasize recommendation for use of new NOAA Atlas 14 IDF tool.
Index 819.5(c)	**Transfer of Data, Page 810-21** Eliminate reference to old hydrology tool, intensity-duration-frequency IDF-32, and emphasize recommendation for use of new NOAA Atlas 14 IDF tool.
Index 819.6	**Hydrologic Computer Programs, Page 810-21** Eliminate reference to old hydrology tool, intensity-duration-frequency IDF-32, and emphasize recommendation for use of new NOAA Atlas 14 IDF tool.
Figure 819.7C	**San Bernardino County Hydrograph for Desert Areas, Page 810-35** Titles for each axis on graph were duplicated for clarity.
Index 852.1(2)	**Indirect Design Strength Requirements, Page 850-1** The load strength of reinforced concrete pipe was further described as the "D" load strength under the new sub index name.
Index 852.2	**Concrete Box and Arch Culverts, Page 850-3** Cast-in-place concrete pipe guidance previously provided in this Index was eliminated. Existing concrete box and arch culvert guidance replaced this index with updates. The Department eliminated this product as well as ribbed HDPE pipe and ribbed PVD pipe products from the standards due to lack of applicability to highway construction in California or cessation of manufacture.
Index 852.3	**Corrugated Steel Pipe, Steel Spiral Rib Pipe and Pipe Arches, Page 850-3** See summary for Index 852.2 changes.
Index 852.4	**Corrugated Aluminum Pipe, Aluminum Spiral Rib Pipe and Pipe Arches, Page 850-6** See summary for Index 852.2 changes.
Index 852.6	**Plastic Pipe, Page 850-9** See summary for Index 852.2 changes.

Index 853.2	**Caltrans Host Pipe Structural Philosophy, Page 850-10** The statement "If rehabilitation of the culvert is determined to be a feasible option" was added to guidance directed at reinforcing the structure integrity of the host pipe.
Table 853.1B	**Guide for Plastic Pipeliner Selection in Abrasive Conditions to Achieve 50 Years of Maintenance-Free Service Life, Page 850-13** Footnotes added to Table.
Index 854.1(1)(d)	**Joint Strength Properties, Page 850-15** Revised reference to reflect name change in Standard Specifications, Section 61 "Culvert and Drainage Pipe Joints".
Index 854.1(2)	**Joint Leakage, Page 850-16** Reference to Standard Specification Section 88-1.03 is no longer relevant and was deleted.
Index 855.2	**Abrasion, Page 850-19** Clarification was added that increased velocities with minor bedload volumes are regardless of the gradation.
Table 855.2A	**Abrasion Levels and Materials, Page 850-23** Minor edits.
Table 855.4A	**Guide for Protection of Cast-in-Place and Precast Reinforced and Unreinforced Concrete Structures Against Acid and Sulfate Exposure Conditions, Page 850-34** Footnotes were updated and added as superscripts in the Table.
Index 855.5	**Material Susceptibility to Fire, Page 850-36** Additional data and guidance is provided on addressing pipe selection and treatments in fire prone locations.
Topic 856	**Height of Fill, Page 850-36** Multiple changes throughout, primarily related to Departmental conversion to AASHTO LRFD structural design standards. The conversion to AASHTO LRFD is provided primarily via changes to pipe fill height tables, as shown in Tables 856.3A-856.3P, and Table 856.4.
Chapter 870	**Page 870-Throughout** Multiple revisions to update references to the Standard Specifications or Standard Special Provisions were made to ensure conformance with the 2010 Edition of the Standard Plans, Standard Specifications, and Standard Special Provisions. Also, multiple changes to revise nomenclature for Rock Slope Protection Fabric to conform to Section 88 of the Standard Specifications. New Class 8 and Class 10 designations replaced Type A and Type B.
Index 871.3	**Selected References, Page 870-2** Update references to FHWA publications.
Table 873.3A	**Guide for Determining RSP-Class of Outer side Layer, Page 870-29** Minor revisions to improve clarity.

Chapter 880 **Formally Underground Disposal, Pages 880-1 through 880-2**
Deleted, pease remove this page.

Enclosures available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm.

Table of Contents

Table of Contents

CHAPTER 820 - CROSS DRAINAGE

Table of Contents

Table of Contents

Table of Contents

Table of Contents

Table of Contents

Table of Contents

Table of Contents

Topic Number	Subject	Page Number

List of Figures

List of Figures

List of Figures

List of Figures

Figure Number	Subject	Page Number

CHAPTERS 600-670 - PAVEMENT ENGINEERING

CHAPTER 600 – GENERAL ASPECTS

CHAPTER 610 – PAVEMENT ENGINEERING CONSIDERATIONS

CHAPTER 620 – RIGID PAVEMENT

CHAPTER 650 – PAVEMENT DRAINAGE

CHAPTER 660 – BASE AND SUBBASE

CHAPTER 670 – STRUCTURE APPROACH SLABS

CHAPTERS 800-890 - HIGHWAY DRAINAGE DESIGN

CHAPTER 800 - GENERAL ASPECTS

List of Figures

List of Figures

List of Tables

List of Tables

List of Tables

CHAPTER 860 - OPEN CHANNELS

CHAPTER 870 - CHANNEL AND SHORE PROTECTION – EROSION CONTROL

CHAPTER 900 – LANDSCAPE ARCHITECTURE

CHAPTER 1000 - BIKEWAY PLANNING AND DESIGN

- Slow moving equipment could be kept off the freeway.
- Site not accessible to equipment from the freeway.
- Gates necessary for access to facilities outside the freeway right of way that cannot be reached from local streets or roads.

(b) Proposals for locked gates to be used by other public agencies or utility companies must be submitted to the Chief, Division of Design for approval. The submittal should give all the facts justifying approval and comparisons with alternate solutions.

Criteria for justification are generally the same as for gates used exclusively by highway maintenance forces except for parking. Safe and adequate parking is a necessary part of the solution to access by other agencies.

Locked gates to be used by non-utility entities require FHWA approval under any of the following circumstances:

- The gate is on an Interstate route.
- Federal-aid funds participated in the cost of right of way.
- Federal-aid funds participated or may participate in the cost of construction.

When proposals for locked gates requiring FHWA approval are included in the plans for new construction, including landscaping projects, FHWA approval of such gates will be included in FHWA approval of the project PS&E. Subsequent installations requiring FHWA approval will be submitted separately to FHWA by the Division of Design after approval by the Chief, Division of Design.

701.3 Fences on Other Highways

(1) Policy. The State will construct or pay the cost of fences on private property only as a right of way consideration to mitigate damages. State construction of such fences should be limited to:

(a) The reconstruction or replacement of existing fences.

(b) The construction of fences across property that had been previously enclosed by fences.

This policy applies to all private as well as public lands.

(2) Types of Fences. Only Type BW and Type WM fences on either metal or wood posts are to be constructed by the State on highways other than freeways and expressways.

(3) Location of Fences. Fences on other highways are placed along the right of way line inside the abutting property.

Topic 702 - Miscellaneous Traffic Items

702.1 References

(1) Guardrail and Crash Cushions. See Chapter 7 of the Traffic Manual.

(2) Markers. See Part 3 of the California Manual on Uniform Traffic Control Devices (California MUTCD).

(3) Truck Escape Ramps. See Traffic Bulletin No. 24, (1986) and the NCHRP Report 178.

(4) Mailboxes. See the AASHTO Roadside Design Guide, 3rd Edition, Chapter 11, "Erecting Mailboxes on Streets and Highways".

Topic 703 - Special Structures and Installation

703.1 Truck Weighing Facilities

The Division of Traffic Operations coordinates the design and construction of truck weighing facilities with the California Highway Patrol in Sacramento. Typical plans showing geometric details of these facilities are available from the Headquarters Division of Traffic Operations. Districts should refer truck weighing facility maintenance issues to their District maintenance units.

See Index 107.1 for additional details on roadway connections for truck weighing facilities.

703.2 Rockfall Restraining Nets

Rockfall Restraining Nets are protective devices designed to control large rockfall events and prevent rock from reaching the traveled way. The systems consist of rectangular panels of woven wire rope vertically supported by steel posts and designed with frictional brake elements capable of absorbing and dissipating high energies. For additional information on the characteristics and applications for rockfall restraining nets, designers should contact the Division of Engineering Services - Geotechnical Services (DES-GS).

Topic 704 - Contrast Treatment

704.1 Policy

In general, delineation should be composed of the standard patterns discussed in Part 3 of the California MUTCD.

Markings include lines and markings applied to the pavement, raised pavement markers, delineators, object markers, and special pavement treatments.

Contrast treatment is designed primarily to provide a black color contrast with an adjacent white surface. Normally, contrast treatment should be used only in special cases such as the following:

(a) To provide continuity of surface texture for the guidance of drivers through construction areas.

(b) To provide added emphasis on an existing facility where driver behavior has demonstrated that standard signs and markings have proven inadequate.

When contrast treatment is applied, a slurry seal should be used.

See Part 3 of the California MUTCD for additional information on contrast treatment.

Topic 705 - Materials and Color Selection

705.1 Special Treatments and Materials

Special materials or treatments, such as painted concrete, or vinyl-clad fences, are sometimes proposed for aesthetic reasons, or to comply with special requirements.

The following guidelines are to be used for the selection of these items:

(a) Concrete should not be painted unless exceptional circumstances exist, due to the continuing and expensive maintenance required. Concrete subject to unintentional staining should be textured during construction to minimize the visibility of stains, if other methods of controlling stain-producing runoff or dripping cannot be accomplished.

(b) Vinyl-clad fences are sometimes specified for aesthetic reasons. The cost of this material is higher than that of galvanized steel. Special consideration should be given to the life-cycle cost and maintainability of vinyl-clad fencing prior to selection for use. The use of black or green vinyl-clad mesh for access control fencing, safety fencing at the top of retaining walls, and pedestrian overcrossing fencing is acceptable.

705.2 Colors for Steel Structures

Colors for steel bridges and steel sign structures may be green, gray, or neutral tones of brown, tan, or light blue.

Criteria for selection of colors are:

(a) General continuity along any given route.

(b) Coordination of color schemes with adjacent Districts for interdistrict routes.

(c) Requests from local agencies for improvement of aesthetics in their community.

Color selection for steel bridges should be mutually satisfactory to the Division of Engineering Services continuous in the direction of flow and may extend laterally beyond the definite banks to include overflow channels contiguous to the ordinary channel. The term does not include artificial channels such as canals and drains, except natural channels trained or restrained by the works of man. Neither does it include depressions or swales through which surface or errant waters pass.

Watershed. The area that contributes surface water runoff into a tributary system or water course.

Water Table. The surface of the groundwater below which the void spaces are completely saturated.

Waterway. (1) That portion of a watercourse which is actually occupied by water (2) A navigable inland body of water.

Wave. (1) An oscillatory movement of water on or near the surface of standing water in which a succession of crests and troughs advance while particles of water follow cyclic paths without advancing. (2) Motion of water in a flowing stream so as to develop the surficial appearance of a wave.

Wave Height. The vertical distance between a wave crest and the preceding trough.

Wave Length. The horizontal distance between similar points on two successive waves (e.g., crest to crest or trough to trough), measured in the direction of wave travel.

Wave Period. The time in which a wave crest travels a distance equal to one wave length. Can be measured as the time for two successive wave crests to pass a fixed point.

Weephole. A hole in a wall, invert, apron, lining, or other solid structure to relieve the pressure of groundwater.

Weir. A low overflow dam or sill for measuring, diverting, or checking flow.

Well. (1) Artificial excavation for withdrawal of water from underground storage. (2) Upward component of velocity in a stream.

Wetland. Those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

Windbreak. Barrier fence or trees to break or deflect the velocity of wind.

Windwave. A wave generated and propelled by wind blowing along the water surface.

Young. Immature, said of a stream on a steep gradient actively scouring its bed toward a more stable grade.

Topic 807 - Selected Drainage References

807.1 Introduction

Hydraulic and drainage related reference publications listed are grouped as to source.

807.2 Federal Highway Administration Hydraulic Publications

Copies of publications identified with an NTIS or GPO number may be ordered as follows:

NTIS - Send a check to:

> National Technical Information Service
> 5285 Port Royal Road
> Springfield, VA 22161
> (703) 487-4650

GPO - Send a check to:

> Superintendent of Documents
> Government Printing Office
> Washington, D.C. 20402
> (202) 783-3238

(1) Hydraulic Engineering Circulars (HEC).

HEC No.	Title	Date	FHWA # NTIS #
9	Debris-Control Structures	2005	IF-04-016
11	Design of Riprap Revetment	2000	IF-00-022
14	Hydraulic Design of Energy Dissipators for Culverts and Channels	2006	NHI-06-086
15	Design of Roadside Channels with Flexible Linings	2005	IF-15-114
17	The Design of Encroachments on Flood Plains Using Risk Analysis	1981	EPD-86-112 PB86-182110/AS
18	Evaluating Scour at Bridges	2001	NHI-01-001
20	Stream Stability at Highway Structures	2001	NHI-01-002
21	Bridge Deck Drainage Systems	1993	SA-92-010 PB94-109584
22	Urban Drainage Design Manual	2009	NHI-10-009
23	Bridge Scour and Stream Instability Countermeasures	2009	NHI-09-111 NHI-09-012
24	Highway Stormwater Pump Station Design	2001	NHI-01-007
25	Highways in the Coastal Environment	2008	NHI-07-096
26	Culvert Designer Aquatic Organism Passage	2010	HIF-11-008

(2) Hydraulic Design Series (HDS).

HDS No.	Title	Date	FHWA # NTIS #
1	Hydraulics of Bridge Waterways	1978	EPD-86-101 PB86-181708/AS
2	Highway Hydrology	2002	NHI-02-001
3	Design Charts for Open-Channel Flow	1961	EPD-86-102 PB86-179249/AS
4	Introduction to Highway Hydraulics	2008	NHI-08-090
5	Hydraulic Design of Highway Culverts (GPO 050-001-00298-1)	2005	NHI-01-020
6	River Engineering for Highway Encroachments	2001	NHI-01-004

(3) Implementation Publications.

Title	Date	FHWA # NTIS #
Structural Design Manual for Improved Inlets and Culverts	1983	IP-83-6 PB84-153485
Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains	1984	TS-84-204 PB84-242585
Culvert Inspection Manual	1986	IP-86-2 PB87-151809

(4) Publications on CD-ROM.

Title	Date	FHWA # NTIS #
HDS-5 Hydraulic Design of Highway Culverts	(CDROM) v 1.00 1996	SA-96-080
Installation and User's Guide	1996	SA-96-081

Table 808.1

Summary of Related Computer Programs and Web Applications

	Storm Drains	Hydrology	Water Surface Profiles	Culverts	Roadside /Median Channels	Pavement Drainage	Pond Routing
FHWA Hydraulic Toolbox					x	x	
TR-55		x					
HEC-HMS [(2)]		x					x
HY-8				x			
HEC-RAS [(1)]			x				
FESWMS			x				
HDS No 5: CD				x			
WMS		x		x			x
IDF 2000		x					
NOAA Atlas 14		x					
USGS StreamStats		x					
AutoDesk Civil 3D/Hydraflow	x	x				x	x

NOTES:

(1) The data that was used by FEMA to establish water surface elevations (usually HEC-2) must be used to develop a duplicate effective model for FEMA floodplain analysis. For more information contact FEMA or the Local Agency.

(2) HEC-1 has been superseded by HEC-HMS by the U.S. Army Corps of Engineers.

Special circumstances may dictate the use of alternative methods/programs. Any such use should be performed under direction and with approval of the District Hydraulics Engineer.

The validity of hydrologic analysis using observed historical highwater marks may be affected by aggradation or degradation of the streambed. The effects of aggradation and degradation are important considerations in selecting an effective drainage system design to protect highways and adjacent properties from damage. For more information refer to the FHWA report entitled, "Stream Channel Degradation and Aggradation: Analysis of Impact to Highway Crossings".

813.8 Debris

The quantity and size of solid matter carried by a stream may affect the hydrologic analysis of a drainage basin. Bulking due to mud, suspended sediment and other debris transported by storm runoff may significantly increase the volume of flow, affect flow characteristics, and can be a major consideration in the hydraulic design of drainage structures. In particular, bulking factors are typically a consideration in determining design discharges for facilities with watersheds that are located within mountainous regions subject to fire and subsequent soil erosion, or in arid regions when the facility is in the vicinity of alluvial fans (see Index 872.3(5) for special considerations given to highways located across desert washes).

Debris control methods, structures, and design considerations are discussed in Topic 822, Debris Control.

The District Hydraulics Engineer should be consulted for any local studies that may be available. If both stream gage data and local studies are available, a determination of whether post-fire peak flows are included within the data record should be made. Consideration should be given to treating a significant post-fire peak as the design discharge in lieu of the peak discharge obtained through gage analysis for a given probability flood event. Records of stream discharge from burned and long-unburned (unburned for 40 years or more years) areas have showed peak discharge increases from 2 to 30 times in the first year after burning. In mountainous regions subject to fire with no local studies available, the U.S. Forest Service should be contacted for fire history in order to determine if there is a significant post-fire peak within the stream records.

Topic 814 - Meteorological Characteristics

814.1 General

Meteorology is the science dealing with the earth's atmosphere, especially the weather. As applied to hydrology for the highway designer the following elements of meteorological phenomena are considered the more important factors affecting runoff and flood predictions.

814.2 Rainfall

Rainfall is the most common factor used to predict design discharge. Unfortunately, due to the many interactive factors involved, the relationship between rainfall and runoff is not all that well defined. Intuitively, engineers know and studies confirm, that runoff increases in proportion to the rainfall on a drainage basin. Highway design engineers are cautioned about assuming that a given frequency storm always produces a flood of the same frequency. There are analytical techniques for ungaged watersheds that are based on this assumption. A statistical analysis of extensive past rainfall records should be made before such a correlation is accepted.

Rainfall event characteristics which are important to highway drainage design are:

- Intensity (rate of rainfall)
- Duration (time rainfall lasts)
- Frequency (statistical probability of how often rainfall will occur)
- Time Distribution (intensity hyetograph)
- Storm Type (orographic, convective or cyclonic)
- Storm Size (localized or broad areal extent)
- Storm Movement (direction of storm)

814.3 Snow

Much of the precipitation that falls in the mountainous areas of the state falls as frozen water in the form of snow, hail, and sleet. Since frozen precipitation cannot become part of the runoff until melting occurs it is stored as snowpack until thawed by warmer weather.

Rain upon an accumulation of snow can cause a much higher peak discharge than would occur from rainfall alone. The parameters of snow which may need to be considered in quantifying peak flood runoff are:

- Mean annual snowfall
- Water content of snowpack
- Snowmelt rate

814.4 Evapo-transpiration

Evaporation and transpiration are two natural processes by which water reaching the earth's surface is returned to the atmosphere as vapor. The losses due to both phenomena are important to long term hydrology and water balance in the watershed and are usually ignored in the hydrologic analysis for the design of highway drainage facilities.

814.5 Tides and Waves

The combined effect of upland runoff and tidal action is a primary consideration in the design of highway drainage structures and shore protection facilities along the coastlines, on estuaries, and in river delta systems.

The time and height of high and low water caused by the gravitational attraction of the sun and moon upon the earth's oceans are precisely predictable. Information on gravitational tides and tidal bench marks for the California Coastline is available from:

State Lands Commission
NOS Marine Boundary Program
1807 13th Street
Sacramento, CA 95814

Or from the following web-site:
http://co-ops.nos.noaa.gov/bench.html.

One of the most devastating forces affecting the coastline occurs when an astronomical high tide and a storm of hurricane proportion arrive on the land at the same time. This is also true of the effect of a tsunami. A tsunami is a wave caused by an earthquake at sea. If shore protection were designed to withstand the forces of a tsunami, it would be extremely costly to construct. Since it would be so costly and the probability of occurrence is so slight, such a design may not be justified.

Wind-waves directly affect coastal structures and cause dynamic changes in coastal morphology. The U.S. Corps of Engineers collects and publishes data which may be used to predict size of Pacific Coast wind-waves. Information pertaining to the California coastline from the Mexican border north to Cape San Martin can be obtained from:

U.S. Army Corps of Engineers
Los Angeles District
P.O. Box 2711
Los Angeles, CA 90053
(213) 688-5400

For information from Cape San Martin to the Oregon border from:

U.S. Army Corps of Engineers
San Francisco District
211 Main Street
San Francisco, CA 94105
(415) 556-3582

Wind-waves are also generated on large inland bodies of water and their effect should be considered in the design of shoreline highway facilities.

Topic 815 - Hydrologic Data

815.1 General

The purpose for which a hydrologic study is to be made will determine the type and amount of hydrologic data needed. The accuracy necessary for preliminary studies is usually not as critical as the desirable accuracy of a hydrologic analysis to be used for the final design of highway drainage structures. If data needs can be clearly identified, data collection and compilation efforts can be tailored to the importance of the project.

Data needs vary with the methods of hydrologic analysis. Highway engineers should remember that there is no single method applicable to all design problems. They should make use of whatever hydrologic data that has been developed by others whenever it is available and applicable to their needs.

Frequently there is little or no data available in the right form for the project location. For a few locations in the State, so much data has been compiled that it is difficult to manage, store, and retrieve the information that is applicable to the project site.

815.2 Categories

For most highway drainage design purposes there are three primary categories of hydrologic data:

(1) *Surface Water Runoff.* This includes daily and annual averages, peak discharges, instantaneous values, and highwater marks.

(2) *Precipitation.* Includes rainfall, snowfall, hail, and sleet.

(3) *Drainage Basin Characteristics.* Adequate information may not be readily available but can generally be estimated or measured from maps, field reviews or surveys. See Topic 812 for a discussion of basin characteristics.

Other special purpose categories of hydrologic data which may be important to specific problems associated with a highway project are:

- Sediment and debris transport
- Snowpack variations
- Groundwater levels and quantity
- Water quality

815.3 Sources

Hydrologic data necessary for the design of cross drainage (stream crossings) are usually obtained from a combination of sources.

(1) *Field Investigations.* A great deal of the essential information can only be obtained by visiting the site. Except for extremely simple designs or the most preliminary analysis, a field survey or site investigation should always be made.

To optimize the amount and quality of the hydrologic data collected the field survey should be well planned and conducted by an engineer with general knowledge of drainage design. Data collected are to be documented. When there is reason to believe that a potential for significant risks or impacts associated with the design of drainage facilities may exist, a written report with maps and photographs may be necessary. (See Topic 804 for Floodplain Encroachments.) Appended to HDS No. 2 is a checklist for drainage studies and reports which may be a useful guide in the conduct of hydrologic studies. Typical data collected in a field survey are:

- Highwater marks
- Performance and condition of existing drainage structures
- Stream alignment
- Stream stability and scour potential
- Land use and potential development
- Location and nature of physical and cultural features
- Vegetative cover
- Upstream constraints on headwater elevation
- Downstream constraints
- Debris potential

(2) *Federal Agencies.* The following agencies collect and disseminate stream flow data:

- Geological Survey (USGS)
- Corps of Engineers (COE)
- Bureau of Reclamation (USBR)
- Soil Conservation Service (SCS)
- Forest Service (USFS)
- Bureau of Land Management (BLM)
- Federal Emergency Management Agency (FEMA)
- Environmental Protection Agency (EPA)

The USGS is the primary federal agency charged with collecting and maintaining water related data. Stream-gaging station data and other water related information collected by the USGS is published in Water Supply Papers and through the USGS Office of Surface Water website.

(3) *State Agencies.* The primary state agency collecting stream-gaging and precipitation (rain-gage and snowfall) data is the California Department of Water Resources (DWR).

(4) *Local Agencies.* Entities such as cities, counties, flood control districts, or local improvement districts study local drainage conditions and are often a valuable source of hydrologic data.

(5) *Private Sector.* Water using industries or utilities, railroads and local consultants frequently have pertinent hydrologic records and studies available.

815.4 Stream Flow

Once surface runoff water enters into a stream, it becomes "stream flow". Stream flow is the only portion of the hydrologic cycle in which water is so confined as to make possible reasonably accurate measurements of the discharges or volumes involved. All other measurements in the hydrologic cycle are, at best, only inadequate samples of the whole.

The two most common types of stream flow data are:

- Gaging Stations - data generally based on recording gage station observations with detailed information about the stream channel cross section. Current meter measurements of transverse channel velocities are made to more accurately reflect stream flow rates.
- Historic - data based on observed high water mark and indirect stream flow measurements.

Stream flow data are usually available as mean daily flow or peak daily flow. Daily flow is a measurement of the rate of flow in cubic feet per second (CFS) for the 24-hour period from midnight to midnight.

"Paleoflood" (ancient flood) data has been found useful in extending stream gaging station records. (See Topic 817 for further discussion on measuring stream flow)

815.5 Rainfall

Rainfall data are collected by recording and non-recording rain gages. Rainfall collected by vertical cylindrical rain gages of about 8 inches in diameter is designated as "point rainfall".

Regardless of the care and precision used, rainfall measurements from rain gages have inherent and unavoidable shortcomings. Snow and wind problems frequently interrupt rainfall records. Extreme rainfall data from recording rain gage charts are generally underestimated.

Rain gage measurements are seldom used directly by highway engineers. The statistical analysis which must be done with precipitation measurements is nearly always performed by qualified hydrologists and meteorologists such as those employed by the Department of Water Resources (DWR). The intensity-duration-frequency (IDF) tables and curves are the products of rainfall measurement analyses which have direct application to highway drainage design.

815.6 Adequacy of Data

All hydrologic data that has been collected must be evaluated and compiled into a usable format. Experience, knowledge and judgment are an important part of data evaluation. It must be ascertained whether the data contains inconsistencies or other unexplained anomalies which might lead to erroneous calculations and conclusions that could result in the over design or under design of drainage structures.

Topic 816 - Runoff

816.1 General

The process of surface runoff begins when precipitation exceeds the requirements of:

- Vegetal interception.
- Infiltration into the soil.
- Filling surface depressions (puddles, swamps and ponds). As rain continues to fall, surface waters flow down slope toward an established channel or stream.

816.2 Overland Flow

Overland flow is surface waters which travel over the ground as sheet flow, in rivulets and in small channels to a watercourse.

816.3 Subsurface Flow

Waters which move laterally through the upper soil surface to streams are called "interflow" or "subsurface flow". For the purpose of highway drainage hydrology, where peak design discharge (flood peaks) are the primary interest, subsurface flows are considered to be insignificant. Subsurface flows travel slower than overland flow.

While groundwater and subsurface water may be ignored for runoff estimates, their detrimental effect upon highway structural section stability cannot be overstated. See Chapter 840, Subsurface Drainage.

816.4 Detention and Retention

Water which accumulates and ponds in low points or depressions in the soil surface with no possibility for escape as runoff is in retention storage. Where water is moving over the land it is in detention storage. Detained water, as opposed to retained water, contributes to runoff.

816.5 Flood Hydrograph and Flood Volume

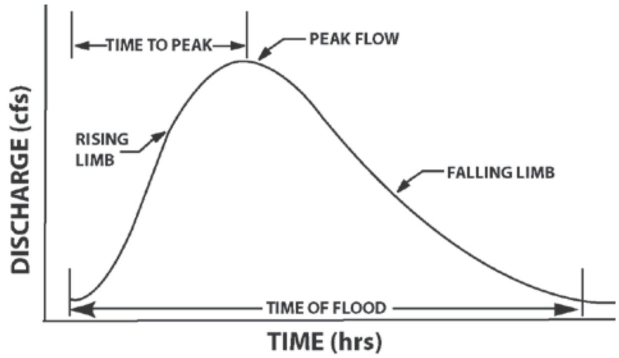
In response to a rainstorm the quantity of water flowing in a stream increases. The water level rises and may continue to do so after rainfall ceases. The response of an affected stream, during and after a storm event, can be pictured by plotting discharge against time to produce a flood hydrograph. The principal elements of a typical flood hydrograph are shown in Figure 816.5

Flood volume is the area under the flood hydrograph. Although flood volume is not normally a consideration in the design of highway drainage facilities, it is occasionally used in the hydrologic analysis for other design parameters.

Information on flood hydrographs and methods to estimate the hydrograph may be found in Chapters 6, 7 and 8 of HDS No. 2, Hydrology.

Figure 816.5

Typical Flood Hydrograph



816.6 Time of Concentration (Tc) and Travel Time (Tt)

Time of concentration is defined as the time required for storm runoff to travel from the hydraulically most remote point of the drainage basin to the point of interest.

An assumption made in some of the hydrologic methods for estimating peak discharge, such as the Rational and NRCS Methods (Index 819.2), is that maximum flow results when rainfall of uniform intensity falls over the entire watershed area and the duration of that rainfall is equal to the time of concentration. Time of concentration (T_C) is typically the cumulative sum of three travel times, including:

- Sheet flow
- Shallow concentrated flow
- Channel flow

For all-paved watersheds (e.g., parking lots, roadway travel lanes and shoulders, etc.) it is not necessary to calculate a separate shallow concentrated flow travel time segment. Such flows will typically transition directly from sheet flow to channel flow or be intercepted at inlets with either no, or inconsequential lengths of, shallow concentrated flow.

In many cases a minimum time of concentration will have to be assumed as extremely short travel times will lead to calculated rainfall intensities that are overly conservative for design purposes. For all-paved areas it is recommended that a minimum time of concentration of 5 minutes be used. For rural or undeveloped areas, it is recommended that a minimum T_C of 10 minutes be used for most situations. However, for slopes steeper than 1V:10H, or where there is limited opportunity for surface storage, a T_C of 5 minutes should be assumed.

Designers should be aware that maximum runoff estimates are not always obtained using rainfall intensities determined by the time of concentration for the total area. Peak runoff estimates may be obtained by applying higher rainfall intensities from storms of short duration over a portion of the watershed.

(1) Sheet flow travel time. Sheet flow is flow of uniform depth over plane surfaces and usually occurs for some distance after rain falls on the ground. The maximum flow depth is usually less than 0.8 inches - 1.2 inches. For unpaved areas, sheet flow normally exists for a distance less than 80 feet - 100 feet. An upper limit of 300 feet is recommended for paved areas.

A common method to estimate the travel time of sheet flow is based on kinematic wave theory and uses the Kinematic Wave Equation:

$$T_t = \frac{0.93L^{3/5}n^{3/5}}{i^{2/5}S^{3/10}}$$

where

T_t = travel time in minutes.

L = Length of flow path in feet.

S = Slope of flow in feet per feet.

n = Manning's roughness coefficient for sheet flow (see Table 816.6A).

i = Design storm rainfall intensity in inches per hour.

If T_t is used (as part of T_C) to determine the intensity of the design storm from the IDF curves, application of the Kinematic Wave Equation becomes an iterative process: an assumed value of T_t is used to determine i from the IDF curve; then the equation is used to calculate a new value of T_t which in turn yields an updated i. The process is repeated until the calculated T_t is the same in two successive iterations.

To eliminate the iterations, use the following simplified form of the Manning's kinematic solution:

$$T_t = \frac{0.42L^{4/5}n^{4/5}}{P_2^{1/2}s^{2/5}}$$

where P_2 is the 2-year, 24-hour rainfall depth in inches (ref. NOAA Atlas 2, Volume XI or use either of the following web site addresses; http://www.wrcc.dri.edu/pcpnfreq.html or, http://www.nws.noaa.gov/oh/hdsc/noaaatlas2.htm).

The use of flow length alone as a limiting factor for the Kinematic wave equation can lead to circumstances where the underlying assumptions are no longer valid. Over prediction of travel time can occur for conditions with significant amounts of depression storage, where there is high Manning's n-values or for flat slopes. One study suggests that the upper limit of applicability of the Kinematic wave equation is a function of flow length, slope and Manning's roughness coefficient. This study used both field and laboratory data to propose an upper limit of 100 for the composite parameter of $nL/s^{1/2}$. It is recommended that this criteria be used as a check where the designer has uncertainty on the maximum flow length to which the Kinematic wave equation can be applied to project conditions.

Where sheet flow travel distance cannot be determined, a conservative alternative is to assume shallow concentrated flow conditions without an independent sheet flow travel time conditions. See Index 816.6(2).

Table 816.6A
Roughness Coefficients For Sheet Flow

Surface Description	*n*
Hot Mix Asphalt	0.011-0.016
Concrete	0.012-0.014
Brick with cement mortar	0.014
Cement rubble	0.024
Fallow (no residue)	0.05
Grass	
Short grass prairie	0.15
Dense grass	0.24
Bermuda Grass	0.41
Woods(1)	
Light underbrush	0.40
Dense underbrush	0.80

(1) Woods cover is considered up to a height of 1 inch, which is the maximum depth obstructing sheet flow.

(2) *Shallow concentrated flow travel time.* After short distances, sheet flow tends to concentrate in rills and gullies, or the depth exceeds the range where use of the Kinematic wave equation applies. At that point the flow becomes defined as shallow concentrated flow. The Upland Method is commonly used when calculating flow velocity for shallow concentrated flow. This method may also be used to calculate the total travel time for both the sheet flow and the shallow concentrated flow segments under certain conditions (e.g., where use of the Kinematic wave equation to predict sheet flow travel time is questionable, or where the designer cannot reasonably identify the point where sheet flow transitions to shallow concentrated flow).

Average velocities for the Upland Method can be taken directly from Figure 816.6 or may be calculated from the following equation:

$$V = (3.28)\ kS^{1/2}$$

Where S is the slope in percent and k is an intercept coefficient depending on land cover as shown in Table 816.6B.

Table 816.6B
Intercept Coefficients for Shallow Concentrated Flow

Land cover/Flow regime	k
Forest with heavy ground litter; hay meadow	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland	0.152
Short grass pasture	0.213
Cultivated straight row	0.274
Nearly bare and untilled-alluvial fans	0.305
Grassed waterway	0.457

The travel time can be calculated from:

$$T_t = \frac{L}{60\ V}$$

where T_t is the travel time in minutes, L the length in feet, and V the flow velocity in feet per second.

(3) *Channel flow travel time.* When the channel characteristics and geometry are known the preferred method of estimating channel flow time is to divide the channel length by the channel velocity obtained by using the Manning equation, assuming bankfull conditions. See Index 864.3, Open Channel Flow Equations for further discussion of Manning's equation.

Appropriate values for "n", the coefficient of roughness in the Manning equation, may be found in most hydrology or hydraulics text and reference books. Table 864.3A gives some "n" values for lined and unlined channels, gutters, and medians. Procedures for selecting an appropriate hydraulic roughness coefficient may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains". Generally, the channel roughness factor will be much lower than the values for overland flow with similar surface appearance.

Culvert or Storm Drain Flow. Flow velocities in a short culvert are generally higher than they would be in the same length of natural channel and comparable to those in a lined channel. In most cases, including short runs of culvert in the channel, flow time calculation will not materially affect the overall time of concentration (T_c). When it is appropriate to separate flow time calculations, such as for urban storm drains, Manning's equation may be used to obtain flow velocities within pipes.

The TR-55 library of equations for sheet flow, shallow concentrated flow and open channel flow is incorporated into the Watershed Modeling System (WMS) for Time of Concentration Calculations using Triangulated Irregular Networks (TINs) and Digital Elevation Maps (DEMs).

Topic 817 - Flood Magnitude

817.1 General

The determination of flood magnitude from either measurements made during a flood or after peak flow has subsided requires knowledge of open-channel hydraulics and flood water behavior. There are USGS Publications and other technical references available which outline the procedures for measuring flood flow. However, it is only through experience that accurate measurements can be obtained and/or correctly interpreted.

817.2 Measurements

(1) *Direct.* Direct flood flow measurements are those made during flood stage. The area and average velocity can be approximated and the estimated discharge can be calculated, from measurements of flow depth and velocity made simultaneously at a number of points in a cross section.

Discharges calculated from continuous records of stage gaging stations are the primary basis for estimating the recurrence interval or frequency of floods.

(2) *Indirect.* Indirect flood flow measurements are those made after the flood subsides. From channel geometry measurements and high water marks the magnitude of a flood can be calculated using basic open channel hydraulic equations given in Chapter 860. This method of determining flood discharges for given events is a valuable tool to the highway engineer possessing a thorough knowledge and understanding of the techniques involved.

Topic 818 - Flood Probability And Frequency

818.1 General

The estimation of peak discharges of various recurrence intervals is the most common and important problem encountered in highway engineering hydrology. Since the hydrology for the sizing of highway drainage facilities is concerned with future events, the time and magnitude of which cannot be precisely forecast, the highway engineer must resort to probability statistics to define the design discharge.

Modern hydrologists tend to define floods in terms of probability, as expressed in percentage rather than in terms of return period (recurrence interval). Return period, the "N-year flood", and probability (p) are reciprocals, that is, $p = 1/N$. Therefore, a flood having a 50-year return frequency (Q_{50}) is now commonly expressed as a flood with the probability of recurrence of 0.02 (2 percent chance of being exceeded) in any given year.

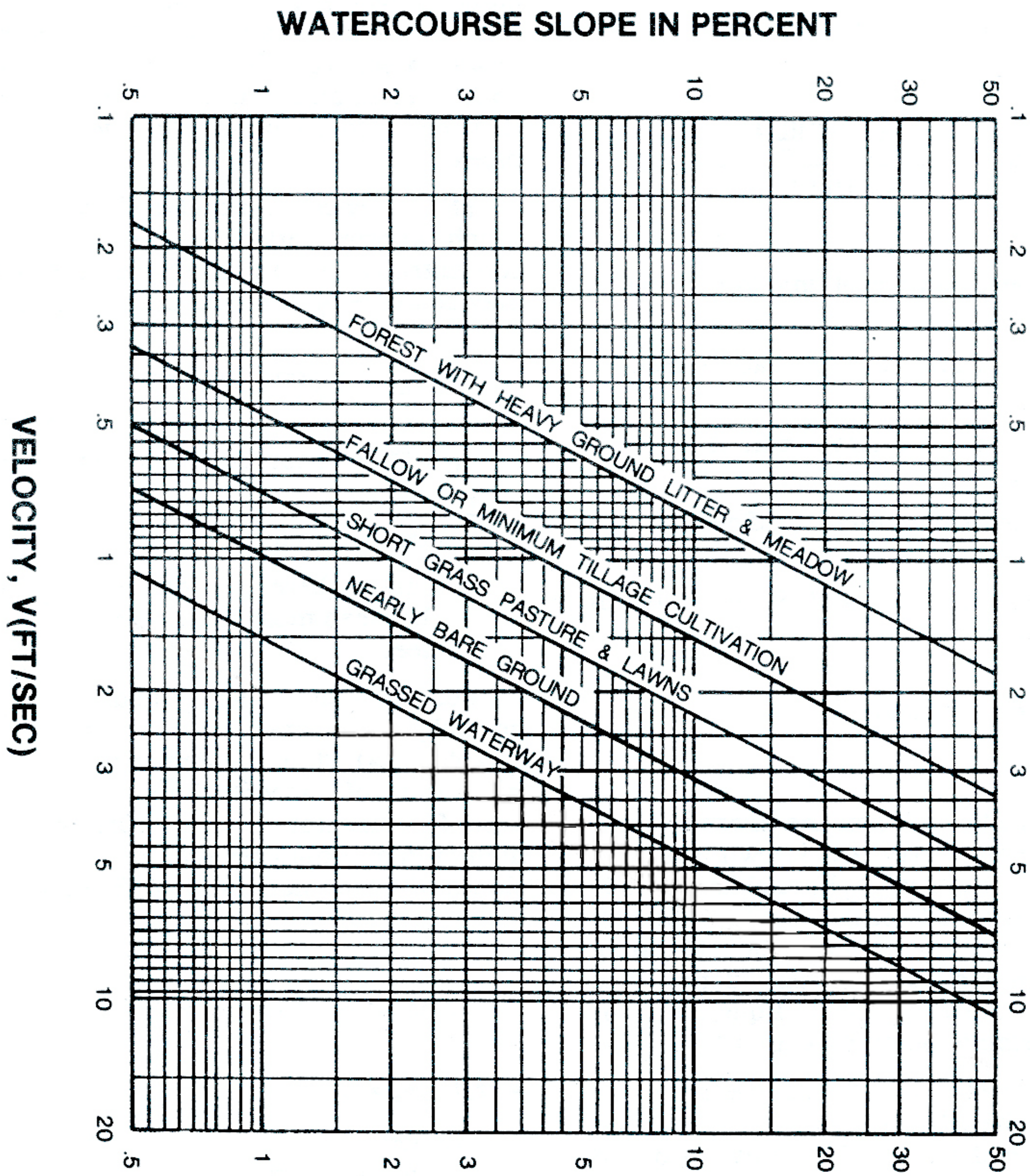
There are certain other terminologies which are frequently used and understood by highway engineers but which might have a slight variation in meaning to other engineering branches. For convenience and example, the following definition of terms have been excerpted from Topic 806, Definition of Drainage Terms.

(1) *Base Flood.* "The flood or tide having a 1 percent chance of being exceeded in any given year". The "base flood" is commonly used as the standard flood in Federal insurance studies and has been adopted by many agencies for flood hazard analysis to comply with regulatory requirements. See Topic 804, Floodplain Encroachments.

(2) *Overtopping Flood.* "The flood described by the probability of exceedance and water surface elevation at which flow occurs over the

Figure 816.6

Velocities for Upland Method of Estimating Travel Time for Shallow Concentrated Flow



highway, over the watershed divide, or through structure(s) provided for emergency relief". The "overtopping flood" is of particular interest to highway drainage engineers because it may be the threshold where the relatively low profile of the highway acts as a flood relief mechanism for the purpose of minimizing upstream backwater damages.

(3) *Design Flood.* "The peak discharge (when appropriate, the volume, stage, or wave crest elevation) of the flood associated with the probability of exceedance selected for the design of a highway encroachment". Except for the rare situation where the risks associated with a low water crossing are acceptable, the highway will not be inundated by the "design flood".

(4) *Maximum Historical Flood.* "The maximum flood that has been recorded or experienced at any particular highway location". This information is very desirable and where available is an indication that the flood of this magnitude may be repeated at the project site. Hydrologic analysis may suggest that the probability for recurrence of the "maximum historical flood" is very small, less than 1 percent. Nevertheless consideration should be given to sizing drainage structures to convey the "maximum historical flood".

(5) *Probable Maximum Flood.* "The flood discharge that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region". The "probable maximum flood" is generally not applicable to highway projects. The possibility of a flood of such rare magnitude, as used by the Corps of Engineers, is applicable to projects such as major dams, when consideration is to be given to virtually complete security from potential floods.

818.2 Establishing Design Flood Frequency

There are two recognized alternatives to establishing an appropriate highway drainage design frequency. That is, by policy or by economic analysis. Both alternatives have merit and may be applied exclusively or jointly depending upon general conditions or specific constraints.

Application of traditional predetermined design flood frequencies implies that an acceptable level of risk was considered in establishing the design standard. Modern design concepts, on the other hand, recommend that a range of peak flows be considered and that the design flood be established which best satisfies the specific site conditions and associated risks. A preliminary evaluation of the inherent flood-related risks to upstream and downstream properties, the highway facility, and to the traveling public should be made. This evaluation will indicate whether a predetermined design flood frequency is applicable or additional study is warranted.

Highway classification is one of the most important factors, but not the sole factor, in establishing an appropriate design flood frequency. Due consideration should be given to all the other factors listed under Index 801.5. If the analysis is correct, the highway drainage system will occasionally be overtaxed. The alternative of accommodating the worst possible event that could happen is usually so costly that it may not be justified.

Highway engineers should understand that the option to select a predetermined design flood frequency is generally only applicable to new highway locations. Because of existing constraints, the freedom to select a prescribed design flood frequency may not exist for projects involving replacement of existing facilities. Caltrans policy relative to up-grading of existing drainage facilities may be found in Index 803.3.

Although the procedures and methodology presented in HEC 17, Design of Encroachments on Flood Plains Using Risk Analysis, are not fully endorsed by Caltrans, the circular is an available source of information on the theory of "least total expected cost (LTEC) design". Highway engineers are cautioned about applying LTEC methodology and procedures to ordinary drainage design problems. The Headquarters Hydraulics Engineer in the Division of Design should be consulted before committing to design by the LTEC method since its use can only be justified and recommended under extra-ordinary circumstances.

Topic 819 - Estimating Design Discharge

819.1 Introduction

Before highway drainage facilities can be hydraulically designed, the quantity of run-off (design Q) that they may reasonably be expected to convey must be established. The estimation of peak discharge for various recurrence intervals is therefore the most important, and often the most difficult, task facing the highway engineer. Refer to Table 819.5A for a summary of methods for estimating design discharge.

In Topic 819, various design recommendations are given for both general and region-specific areas of California.

819.2 Empirical Methods

Because the movement of water is so complex, numerous empirical methods have been used in hydrology. Empirical methods in hydrology have great usefulness to the highway engineer. When correctly applied by engineers knowledgeable in the method being used and its idiosyncrasies, peak discharge estimates can be obtained which are functionally acceptable for the design of highway drainage structures and other features. Some of the more commonly used empirical methods for estimating runoff are as follows.

(1) *Rational Methods*. Undoubtedly, the most popular and most often misused empirical hydrology method is the Rational Formula:

$$Q = CiA$$

Q = Design discharge in cubic feet per second.

C = Coefficient of runoff.

i = Average rainfall intensity in inches per hour for the selected frequency and for a duration equal to the time of concentration.

A = Drainage area in acres.

Rational methods are simple to use, and it is this simplicity that has made them so popular among highway drainage design engineers. Design discharge, as computed by these methods, has the same probability of occurrence (design frequency) as the frequency of the rainfall used. Refer to Topic 818 for further information on flood probability and frequency of recurrence.

An assumption that limits applicability is that the rainfall is of equal intensity over the entire watershed. Because of this, Rational Methods should be used only for estimating runoff from small simple watershed areas, preferably no larger than 320 acres. Even where the watershed area is relatively small but complicated by a mainstream fed by one or more significant tributaries, Rational Methods should be applied separately to each tributary stream and the tributary flows then routed down the main channel. Flow routing can best be accomplished through the use of hydrographs discussed under Index 816.5. Since Rational Methods give results that are in terms of instantaneous peak discharge and provide little information relative to runoff rate with respect to time, synthetic hydrographs should be developed for routing significant tributary inflows. Several relatively simple methods have been established for developing hydrographs, such as transposing a hydrograph from another hydrologically homogeneous watershed. The stream hydraulic method, and upland method are described in HDS No. 2. These, and other methods, are adequate for use with Rational Methods for estimating peak discharge and will provide results that are acceptable to form the basis for design of highway drainage facilities.

It is clearly evident upon examination of the assumptions and parameters which form the basis of the equation that much care and judgment must be applied with the use of Rational Methods to obtain reasonable results.

- The runoff coefficient "C" in the equation represents the percent of water which will run off the ground surface during the storm. The remaining amount of precipitation is lost to infiltration, transpiration, evaporation and depression storage.

Values of "C" may be determined for un-developed areas from Figure 819.2A by considering the four characteristics of: relief, soil infiltration, vegetal cover, and surface storage.

Some typical values of "C" for developed areas are given in Table 819.2B. Should the basin contain varying amounts of different cover, a weighted runoff coefficient for the entire basin can be determined as:

$$C = \frac{C_1A_2 + C_2A_2 + ...}{A_1 + A_2 + ...}$$

- To properly satisfy the assumption that the entire drainage area contributes to the flow; the rainfall intensity, (i) in the equation expressed in inches per hour, requires that the storm duration and the time of concentration (t_C) be equal. Therefore, the first step in estimating (i) is to estimate (t_C). Methods for determining time of concentration are discussed under Index 816.6.
- Once the time of concentration, (t_C), is estimated, the rainfall intensity, (i), corresponding to a storm of equal duration, may be obtained from available sources such as intensity-duration-frequency (IDF) curves. See Index 819.6 for recommendations regarding IDF curve generating software.

The runoff coefficients given in Figure 819.2A and Table 819.2B are applicable for storms of up to 5 or 10 year frequencies. Less frequent, higher intensity storms usually require modification of the coefficient because infiltration, detention, and other losses have a proportionally smaller effect on the total runoff volume. The adjustment of the rational method for use with major storms can be made by multiplying the coefficient by a frequency factor, C(f). Values of C(f) are given below. Under no circumstances should the product of C(f) times C exceed 1.0.

Frequency (yrs)	C(f)
25	1.1
50	1.2
100	1.25

(2) *Regional Analysis Methods.* Regional analysis methods utilize records for streams or drainage areas in the vicinity of the stream under consideration which would have similar characteristics to develop peak discharge estimates. These methods provide techniques for estimating annual peak stream discharge at any site, gaged or ungaged, for probability of recurrence from 50 percent (2 years) to 1 percent (100 years). Application of these methods is convenient, but the procedure is subject to some limitations.

Regional Flood - Frequency equations developed by the U.S. Geological Survey for use in California are given in Figure 819.2C and Table 819.7A. These equations are based on regional regression analysis of data from stream gauging stations. The equations in Figure 819.2C were derived from data gathered and analyzed through the mid-1970's, while the regions covered by Table 819.7A are reflective of a more recent (1994) study of the Southwestern U.S, which has been supplemented by a 2007 Study of California Desert Region Hydrology. Nomographs and complete information on use and development of this method may be found in "Magnitude and Frequency of Floods in California" published in June, 1977 by the U.S. Department of the Interior, Geological Survey.

The Regional Flood-Frequency equations are applicable only to sites within the flood-frequency regions for which they were derived and on streams with virtually natural flows. For example, the equations are not generally applicable to small basins on the floor of the Sacramento and San Joaquin Valleys as the annual peak data which are the basis for the regression analysis were obtained principally in the adjacent mountain and foothill areas. Likewise, the equations are not directly applicable to streams in urban areas affected substantially by urban development. In urban areas the equations may be used to estimate peak discharge values under natural conditions and then by use of the techniques described in the publication or HDS No. 2, adjust the discharge values to compensate for

Figure 819.2A

Runoff Coefficients for Undeveloped Areas

Watershed Types

	Extreme	High	Normal	Low
Relief	.28 -.35 Steep, rugged terrain with average slopes above 30%	.20 -.28 Hilly, with average slopes of 10 to 30%	.14 -.20 Rolling, with average slopes of 5 to 10%	.08 -.14 Relatively flat land, with average slopes of 0 to 5%
Soil Infiltration	.12 -.16 No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity	.08 -.12 Slow to take up water, clay or shallow loam soils of low infiltration capacity, imperfectly or poorly drained	.06 -.08 Normal; well drained light or medium textured soils, sandy loams, silt and silt loams	.04 -.06 High; deep sand or other soil that takes up water readily, very light well drained soils
Vegetal Cover	.12 -.16 No effective plant cover, bare or very sparse cover	.08 -.12 Poor to fair; clean cultivation crops, or poor natural cover, less than 20% of drainage area over good cover	.06 -.08 Fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	.04 -.06 Good to excellent; about 90% of drainage area in good grassland, woodland or equivalent cover
Surface Storage	.10 -.12 Negligible surface depression few and shallow; drainageways steep and small, no marshes	.08 -.10 Low; well defined system of small drainageways; no ponds or marshes	.06 -.08 Normal; considerable surface depression storage; lakes and pond marshes	.04 -.06 High; surface storage, high; drainage system not sharply defined; large flood plain storage or large number of ponds or marshes

Given An undeveloped watershed consisting of;
1) rolling terrain with average slopes of 5%,
2) clay type soils,
3) good grassland area, and
4) normal surface depressions.

Solution:

Relief	0.14
Soil Infiltration	0.08
Vegetal Cover	0.04
Surface Storage	0.06
C=	0.32

Find The runoff coefficient, C, for the above watershed.

Table 819.2B

Runoff Coefficients for Developed Areas

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries:	0.10 - 0.25
Playgrounds:	0.20 - 0.40
Railroad yard areas:	0.20 - 0.40
Unimproved areas:	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2-7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2-7%	0.18 - 0.25
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs:	0.75 - 0.95

urbanization. Further limitations on the use of USGS Regional Flood-Frequency equations are:

Region	Drainage Area (A) mi^2	Mean Annual Precip (P) in	Altitude Index (H) 1000 ft
(1)North Coast	0.2-3000	19-104	0.2-5.7
(2)Northeast	0.2-25	all	all
Sierra	0.2-9000	7-85	0.1-9.7
Central Coast	0.2-4000	8-52	0.1-2.4
South Coast	0.2-600	7-40	all
(3)South Lahontan-Colorado Desert	N/A	N/A	N/A

Notes:

(1) In the North Coast region use a minimum value of 1 for altitude index (H)

(2) See Index 819.7 for hydrologic procedures for those portions of the Northeast Region classified as desert.

(3) USGS equations not recommended. See Index 819.7

A method for directly estimating design discharges for some gaged and ungaged streams is also provided in HDS No. 2. The method is applicable to streams on or nearby those for which study data are available.

(3) Flood Frequency Analysis

(a) If there are two gaged sites with similar watershed characteristics but one has a short record and the other has a longer record of peak flows, a two-station comparison analysis can be conducted to extend the equivalent length of record at the shorter gaged site.

(b) Flood-frequency relations at sites near gaged sites on the same stream (or in a similar watershed) can be estimated using a ratio of drainage area for the ungaged and gaged sites.

(c) At a gaged site, weighted estimates of peak discharges based on the station flood-frequency relation and the regional regression equations are considered the best estimates of flood frequency and are used to reduce the time-sampling error that may occur in a station flood-frequency estimate.

(d) The flood-frequency flows and the maximum peak discharges at several stations in a region should be used whenever possible for comparison with the peak discharge estimated at an ungaged site using a rainfall-runoff approach or regional regression equation. The watershed characteristics at the ungaged and gaged sites should be similar.

(4) National Resources Conservation Service (NRCS) Methods. The Soil Conservation Service's SCS (former title) National Engineering Handbook, 1972, and their 1975, "Urban Hydrology for Small Watersheds", Technical Release 55 (TR-55), present a graphical method for estimating peak discharge. Most NRCS equations and curves provide results in terms of inches of runoff for unit hydrograph development and are not applicable to the estimation of a peak design discharge unless the design hydrograph is first developed in accordance with prescribed NRCS procedures. NRCS methods and procedures are applicable to drainage areas less than 3 square miles (approx. 2,000 acres) and result in a design hydrograph and design discharge that are functionally acceptable to form the basis for the design of highway drainage facilities.

819.3 Statistical Methods

Statistical methods of predicting stream discharge utilize numerical data to describe the process. Statistical methods, in general, do not require as much subjective judgment to apply as the previously described deterministic methods. They are usually well documented mathematical procedures which are applied to measured or observed data. The accuracy of statistical methods can also be measured quantitatively. However, to assure that statistical method results are valid, the method and procedures used should be verified by an experienced engineer with a thorough knowledge of engineering statistics.

Analysis of gaged data permits an estimate of the peak discharge in terms of its probability or frequency of recurrence at a given site. This is done by statistical methods provided sufficient data are available at the site to permit a meaningful statistical analysis to be made. Water Resources Council Bulletin 17B, 1981, suggests at least 10 years of record are necessary to arrant astatistical analysis. The techniques of inferential statistics, the branch of statistics dealing with the inference of population characteristics, are described in HDS No. 2.

Before data on the specific characteristics to be examined can be properly analyzed, it must be arranged in a systematic manner. Several computer programs are available which may be used to systematically arrange data and perform the statistical computations.

Some common types of data groupings are as follows:

- Magnitude
- Time of Occurrence
- Geographic Location

Several standard frequency distributions have been studied extensively in the statistical analysis of hydrologic data. Those which have been found to be most useful are:

(1) Log-Pearson Type III Distribution. The popularity of the Log-Pearson III distribution is simply based on the fact that it very often fits the available data quite well, and it is flexible enough to be used with a wide variety of distributions. Because of this flexibility, the U.S. Water Resources Council recommends its use by all U.S. Government agencies as the standard distribution for flood frequency studies.

The three parameters necessary to describe the Log-Pearson III distribution are:

- Mean flow
- Standard deviation
- Coefficient of skew

Figure 819.2C
Regional Flood-Frequency Equations

NORTH COAST REGION[2]

$Q_2 = 3.52\ A^{0.90}\ P^{0.89}\ H^{-0.87}$
$Q_5 = 5.04\ A^{0.89}\ P^{0.91}\ H^{-0.35}$
$Q_{10} = 6.21\ A^{0.88}\ P^{0.93}\ H^{-0.27}$
$Q_{25} = 7.64\ A^{0.87}\ P^{0.94}\ H^{-0.17}$
$Q_{50} = 8.57\ A^{0.87}\ P^{0.96}\ H^{-0.08}$
$Q_{100} = 9.23\ A^{0.87}\ P^{0.97}$

NORTHEAST REGION[3,4]

$Q_2 = 22\ A^{0.40}$
$Q_5 = 46\ A^{0.45}$
$Q_{10} = 61\ A^{0.49}$
$Q_{25} = 84\ A^{0.54}$
$Q_{50} = 103\ A^{0.57}$
$Q_{100} = 125\ A^{0.59}$

SOUTH LAHONTAN-COLORADO DESERT REGION[3,4]

$Q_2 = 7.3\ A^{0.30}$
$Q_5 = 53.0\ A^{0.44}$
$Q_{10} = 150\ A^{0.53}$
$Q_{25} = 410.0\ A^{0.63}$
$Q_{50} = 700.0\ A^{0.68}$
$Q_{100} = 1080.0\ A^{0.71}$

SIERRA REGION

$Q_2 = 0.24\ A^{0.88}\ P^{1.58}\ H^{-0.80}$
$Q_5 = 1.20\ A^{0.82}\ P^{1.37}\ H^{-0.64}$
$Q_{10} = 2.63\ A^{0.80}\ P^{1.25}\ H^{-0.58}$
$Q_{25} = 6.55\ A^{0.79}\ P^{1.12}\ H^{-0.52}$
$Q_{50} = 10.4\ A^{0.78}\ P^{1.06}\ H^{-0.48}$
$Q_{100} = 15.7\ A^{0.77}\ P^{1.02}\ H^{-0.43}$

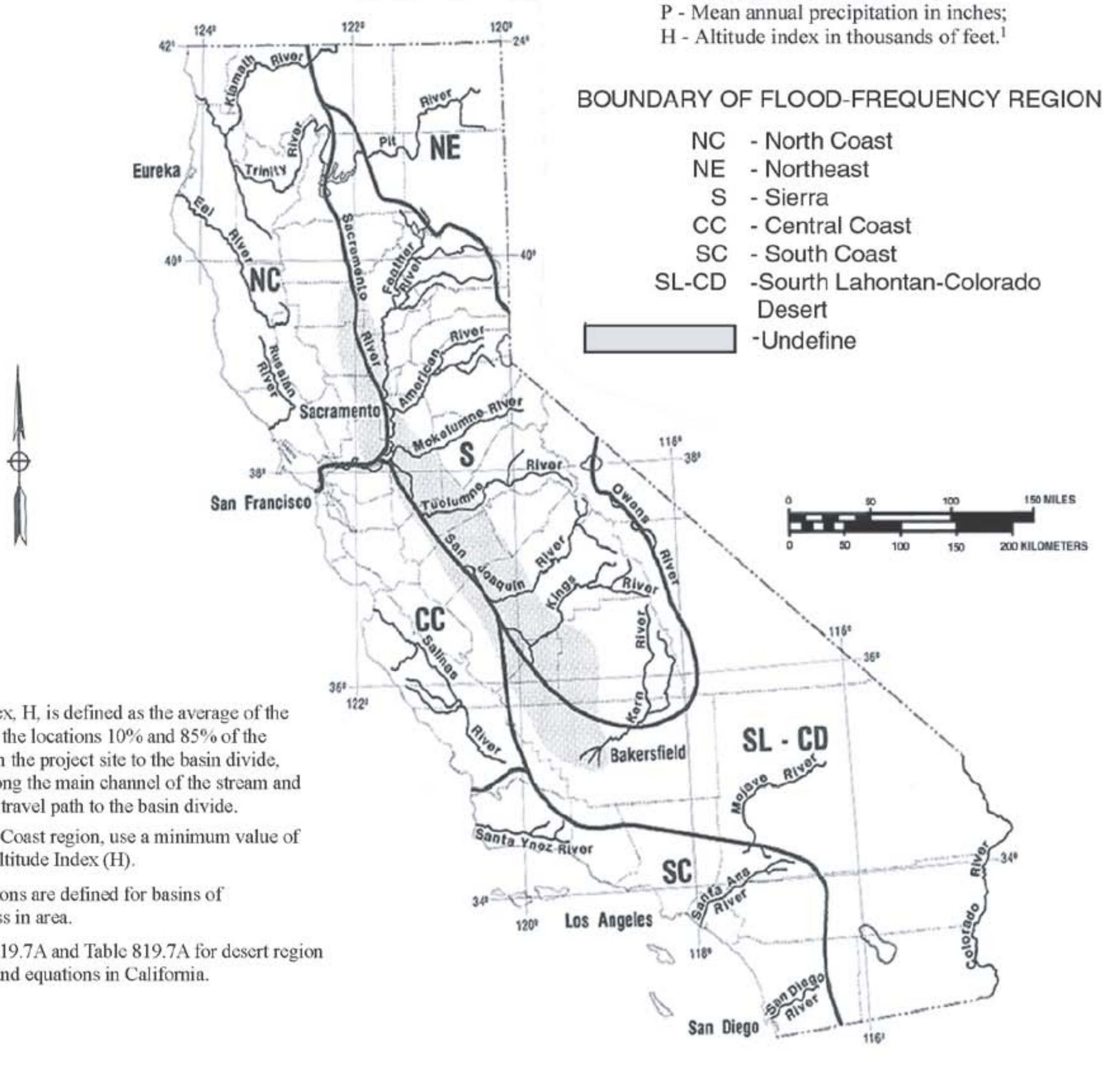
CENTRAL COAST REGION

$Q_2 = 0.0061\ A^{0.92}\ P^{2.54}\ H^{-1.10}$
$Q_5 = 0.118\ A^{0.91}\ P^{1.95}\ H^{-0.79}$
$Q_{10} = 0.583\ A^{0.90}\ P^{1.61}\ H^{-0.64}$
$Q_{25} = 2.91\ A^{0.89}\ P^{1.26}\ H^{-0.50}$
$Q_{50} = 8.20\ A^{0.89}\ P^{1.03}\ H^{-0.41}$
$Q_{100} = 19.7\ A^{0.88}\ P^{0.84}\ H^{-0.33}$

SOUTH COAST REGION

$Q_2 = 0.14\ A^{0.72}\ P^{1.62}$
$Q_5 = 0.40\ A^{0.77}\ P^{1.69}$
$Q_{10} = 0.63\ A^{0.79}\ P^{1.75}$
$Q_{25} = 1.10\ A^{0.81}\ P^{1.81}$
$Q_{50} = 1.50\ A^{0.82}\ P^{1.85}$
$Q_{100} = 1.95\ A^{0.83}\ P^{1.87}$

Q - Peak discharge in CFS, subscript indicates recurrence interval, in years;
A - Drainage area in square miles;
P - Mean annual precipitation in inches;
H - Altitude index in thousands of feet.[1]



NOTES:

1. Altitude Index, H, is defined as the average of the elevations at the locations 10% and 85% of the distance from the project site to the basin divide, measured along the main channel of the stream and the overland travel path to the basin divide.
2. In the North Coast region, use a minimum value of 1.0 for the Altitude Index (H).
3. These equations are defined for basins of 25 mi^2 or less in area.
4. See Figure 819.7A and Table 819.7A for desert region delineation and equations in California.

Log-Pearson III distributions are usually plotted on log-normal probability graph paper for convenience even though the plotted frequency distribution may not be a straight line.

(2) *Log-normal Distribution.* The characteristics of the log-normal distribution are the same as those of the classical normal or Gaussian mathematical distribution except that the flood flow at a specified frequency is replaced with its logarithm and has a positive skew. Positive skew means that the distribution is skewed toward the high flows or extreme values.

(3) *Gumbel Extreme Value Distribution.* The characteristics of the Gumbel extreme value distribution (also known as the double exponential distribution of extreme values) are that the mean flood occurs at the return period of T_r = 2.33 years and that it has a positive skew.

Special probability paper has been developed for plotting log-normal and Gumbel distributions so that sample data, if it is distributed according to prescribed equations, will plot as a straight line.

819.4 Hydrograph Methods

Hydrograph methods of estimating design discharge relate runoff rates to time in response to a design storm. When storage must be considered, such as in reservoirs, natural lakes, and detention basins used for drainage or sediment control, the volume of runoff must be known. Since the hydrograph is a plot of flow rate against time, the area under the hydrograph represents volume. If streamflow and precipitation records are available for a particular design site, the development of the design hydrograph is a straight forward procedure. Rainfall records can be readily analyzed to estimate unit durations and the intensity which produces peak flows near the desired design discharge.

Hydrographs are also useful for determining the combined rates of flow for two drainage areas which peak at different times. Hydrographs can also be compounded and lagged to account for complex storms of different duration and varying intensities. Several methods of developing hydrographs are described in HDS No. 2. For basins without data, two of the most widely used methods described in HDS No. 2 for developing synthetic hydrographs are:

- Unit Hydrograph
- SCS Triangular Hydrograph

Both methods however tend to be somewhat inflexible since storm duration is determined by empirical relations.

819.5 Transfer of Data

Often the highway engineer is confronted with the problem where stream flow and rainfall data are not available for a particular site but may exist at points upstream or in an adjacent or nearby watersheds.

(a) If the site is on the same stream and near a gaging station, peak discharges at the gaging station can be adjusted to the site by drainage area ratio and application of some appropriate power to each drainage area. The USGS may be helpful in suggesting appropriate powers to be used for a specific hydrologic region.

(b) If a design hydrograph can be developed at an upstream point in the same watershed, the procedure described in HDS No. 2 can be used to route the design hydrograph to the point of interest.

(c) IDF curve generating software, such as NOAA's Atlas 14, have internal routines that provide interstation interpolation that accounts not only for distance from gauge stations, but other factors, such as elevation. No additional effort is required by the designer to address distance/location effects.

819.6 Hydrologic Computer Programs

The rapid advancement of computer technology in recent years has resulted in the development of many mathematical models for the purpose of calculating runoff and other hydrologic phenomena. In the hands of knowledgeable and experienced engineers, good computer models are capable of efficiently calculating discharge estimates and other hydrologic results that are far more reliable than those which were obtained by other means. On the other hand, there is a tendency for the inexperienced engineer to accept computer

Table 819.5A
Summary of Methods for Estimating Design Discharge

METHOD	ASSUMPTIONS	DATA NEEDS
Rational	• Small catchment (< 320 acres) • Concentration time < 1 hour • Storm duration >or = concentration time • Rainfall uniformly distributed in time and space • Runoff is primarily overland flow • Negligible channel storage	Time of Concentration Drainage area Runoff coefficient Rainfall intensity
USGS Regional Regression Equations: USGS Water-Resources Investigation 77-21* Improved Highway Design Methods for Desert Storms	• Catchment area limit varies by region • Basin not located on floor of Sacramento or San Joaquin Valleys • Peak discharge value for flow under natural conditions unaffected by urban development and little or no regulation by lakes or reservoirs • Ungaged channel	Drainage area Mean annual precipitation Altitude index
NRCS (TR55)	• Small or midsize catchment (< 3 square miles) • Concentration time range from 0.1-10 hour (tabular hydrograph method limit < 2 hour) • Runoff is overland and channel flow • Simplified channel routing • Negligible channel storage	24-hour rainfall Rainfall distribution Runoff curve number Concentration time Drainage area
Unit Hydrograph (Gaged data) Synthetic Unit Hydrograph SCS Unit Hydrograph S-Graph Unit Hydrograph	• Midsize or large catchment (0.20 square miles to 1,000 square miles) • Uniformity of rainfall intensity and duration • Rainfall-runoff relationship is linear • Duration of direct runoff constant for all uniform-intensity storms of same duration, regardless of differences in the total volume of the direct runoff. • Time distribution of direct runoff from a given storm duration is independent of concurrent runoff from preceding storms • Channel-routing techniques used to connect streamflows	Rainfall hyetograph and direct runoff hydrograph for one or more storm events Drainage area and lengths along main channel to point on watershed divide and opposite watershed centroid (Synthetic Unit Hydrograph)
Statistical (gage data) Log-Pearson Type III Bulletin #17B – U.S. Department of the Interior	• Midsized and large catchments with stream gage data • Appropriate station and/or generalized skew coefficient relationship applied • Channel storage	10 or more years of gaged flood records
Basin Transfer of Gage Data	• Similar hydrologic characteristics • Channel storage	Discharge and area for gaged watershed Area for ungaged watershed

* Magnitude and Frequency of Floods in California

generated output without questioning the reasonableness of the results obtained from a hydrologic viewpoint. Most computer simulation models require a significant amount of input data that must be carefully examined by a competent and experienced user to assure reliable results.

Some hydrologic computer models merely solve empirical hand methods more quickly. Other models are theoretical and solve the entire runoff cycle using mathematical equations to represent each phase of the runoff cycle.

In most simulation models, the drainage area is divided into subareas with similar hydrologic characteristics. A design rainfall is synthesized for each subarea, abstractions removed, and an overland flow routine simulates the movement of surface water into channels. The channels of the watershed are linked together and the channel flow is routed through them to complete the basin's response to the design rainstorm. Simulation models require calibration of modeling parameters using measured historical events to increase their validity.

A summary of personal computer programs is listed in Table 808.1.

Watershed Modeling System (WMS) is a comprehensive environment for hydrologic analysis. It was developed by the Engineering Computer Graphics Laboratory of Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station (WES).

WMS merges information obtained from terrain models and GIS with industry standard hydrologic analysis models such as HEC-1 and TR-55. HY-8 has also been incorporated for culvert design.

Terrain models can obtain geometric attributes such as area, slope and runoff distances. Many display options are provided to aid in modeling and understanding the drainage characteristics of terrain surfaces.

The distinguishing difference between WMS and other applications designed for setting up hydrologic models like HEC-1 and TR-55 is its unique ability to take advantage of digital terrain for hydrologic data development.

WMS uses three primary data sources for model development:

1. Geographic Information Systems (GIS) Data
2. Digital Elevation Models (DEMs) published by the U.S. Geological Survey (USGS) at both 1:24,000 and 1:250,000 for the entire U.S. (the 1:24,000 data coverage is not complete)
3. Triangulated Irregular Networks (TINs)

Two other hydrologic computer programs that are commonly used are the Army Corps of Engineers' HEC-HMS and the National Resources Conservation Service's TR-20 Method.

Other programs include the Caltrans Rainfall Intensity-Duration-Frequency Program, IDF2000, which incorporates the California Department of Water Resources (DWR) short duration precipitation data (See Index 815.3(3)) with an updated station-interpolation routine and GIS mapping capability; and the more recent NOAA Atlas 14 web-based IDF product. The NOAA Atlas 14 product is the preferred IDF tool for State highway projects.

819.7 Region-Specific Analysis

(1) Desert Hydrology

Figure 819.7A shows the different desert regions in California, each with distinct hydrological characteristics that will be explained in this section.

(a) Storm Type

Summer Convective Storms - In the southern desert regions (Owens Valley/Mono Lake, Mojave Desert, Sonoran Desert and the Colorado Desert), the dominant storm type is the local thunderstorm, specifically summer convective storms. These storms are characterized by their short duration, over a relatively small area (generally less than 20 mi^2), and intense rainfall, which may result in flash floods. These summer convective storms may occur at any time during the year, but are most common and intense during the summer. General summer storms can also occur over these desert regions, but are rare, and usually occur from mid-August to early October. The rainfall intensity can vary from heavy rainfall to heavy thunderstorms.

General Winter Storm - In the Antelope Valley and Northern Basin and Range regions, the dominant storm type is the general winter storm. These storms are characterized by their long duration, 6 hours to 12 hours or more, and possibly intermittently for 3 days to 5 days over a relatively large area. General winter storms produce the majority of large peaks in the northern desert areas; the majority of the largest peaks discharge greater than or equal to 20 cfs/mi^2 occurred during the winter and fall months in the Owens Valley/Mono Lake and Northern Basin and Range regions. At elevations above 6,000 ft, much of the winter precipitation falls as snow; however, snowfall doesn't play a significant role in flood-producing runoff in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert). In the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range), more floods from snowmelt occur at lower elevations; more than 50 percent of runoff events occurred in spring, most likely snowmelt, but did not produce large floods.

(b) Regional Regression

Newly developed equations for California's Desert regions are shown on Table 819.7A.

While the regression equations for the Northern Basin and Range region provide more accurate results than previous USGS developed equations, there is some uncertainty associated with them. Therefore, the development of a rainfall-runoff model may be preferable for ungaged watersheds in this region.

(c) Rational Method

The recommended upper limit for California's desert regions is 160 acres (0.25 mi^2).

Table 819.7B lists common runoff coefficients for Desert Areas. These coefficients are applicable for storms with 2-year to 10-year return intervals, and must be adjusted for larger, less frequent storms by multiplying the coefficient by an appropriate frequency factor, C(f), as stated in Index 819.2(1) of this manual. The frequency factors, C(f), for 25-year, 50-year and 100-year storms are 1.1, 1.2 and 1.25, respectively. Under no circumstances should the product of C(f) times the runoff coefficient exceed 1.0. If a value of 1.0 is reached, it is recommended to use the value of 0.95.

(d) Rainfall-Runoff Simulation

A rainfall-runoff simulation approach uses a numerical model to simulate the rainfall-runoff process and generate discharge hydrographs. It has four main components: rainfall; rainfall losses; transformation of effective rainfall; and channel routing.

(1) Rainfall

a. Design Rainfall Criteria

The selection of an appropriate storm duration depends on a number of factors, including the size of the watershed, the type of rainfall-runoff approach and hydrologic characteristics of the study watershed. Watershed sizes are analyzed below and are applied to California's Desert regions in Table 819.7C.

Drainage Areas $\leq$ 20 mi^2 – Drainage areas less than 20 mi^2 are primarily representative of summer convective storms, and usually occur in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert regions). Since these storms usually result in intense rainfall, over a small drainage area and are generally less than 6 hours, it is recommended that a 6-hour local design storm be utilized.

Drainage Areas > 20 mi^2 & $\leq$ 100 mi^2 – For drainage areas between 20 mi^2 and 100 mi^2, the critical storm can be a summer convective storm or a general

Figure 819.7A

Desert Regions in California

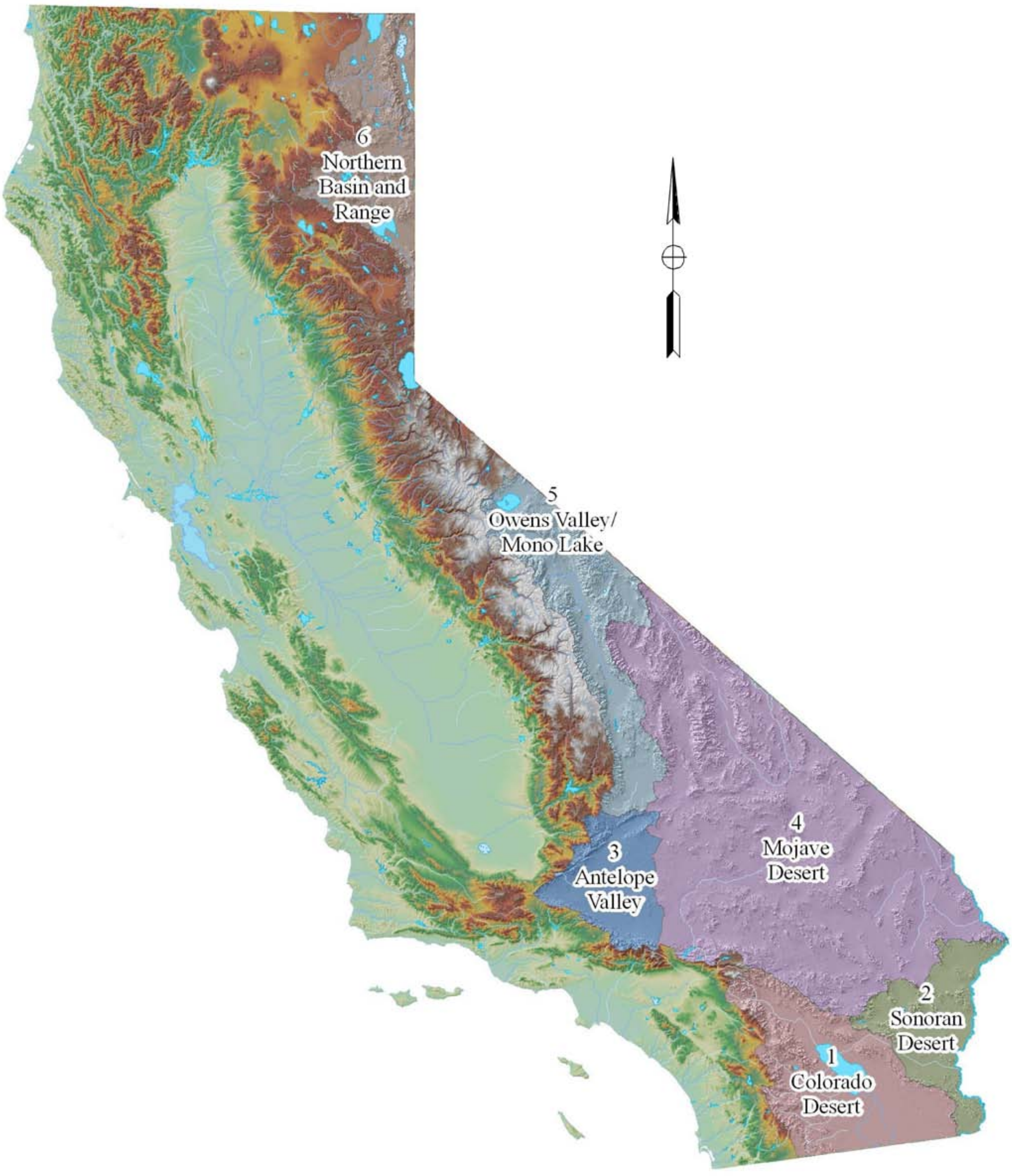


Table 819.7A

Regional Regression Equations for California's Desert Regions

Region(s)	Associated Regression Equations
Colorado Desert Sonoran Desert Antelope Valley Mojave Desert	$Q_2 = 8.57A^{0.5668}$ $Q_5 = 80.32A^{0.541}$ $Q_{10} = 146.33A^{0.549}$ $Q_{25} = 291.04A^{0.5939}$ $Q_{50} = 397.82A^{0.6189}$ $Q_{100} = 557.31A^{0.6619}$
Owens Valley / Mono Lake	$Q_2 = 0.007A^{1.839}\left[\frac{ELEV}{1000}\right]^{1.485}\left[\frac{LAT-28}{10}\right]^{-0.680}$ $Q_5 = 0.212A^{1.404}\left[\frac{ELEV}{1000}\right]^{0.882}\left[\frac{LAT-28}{10}\right]^{-0.030}$ $Q_{10} = 1.28A^{1.190}\left[\frac{ELEV}{1000}\right]^{0.531}\left[\frac{LAT-28}{10}\right]^{0.525}$ $Q_{25} = 9.70A^{0.962}\left[\frac{ELEV}{1000}\right]^{0.107}\left[\frac{LAT-28}{10}\right]^{1.199}$ $Q_{50} = 34.5A^{0.829}\left[\frac{ELEV}{1000}\right]^{-0.170}\left[\frac{LAT-28}{10}\right]^{1.731}$ $Q_{100} = 111A^{0.707}\left[\frac{ELEV}{1000}\right]^{-0.429}\left[\frac{LAT-28}{10}\right]^{2.241}$

Table 819.7A

Regional Regression Equations for California's Desert Regions (Con't)

Northern Basin & Range	$Q_2 = 5.320A^{0.415}\left[\frac{H}{1000}\right]^{0.928}$ $Q_5 = 29.71A^{0.360}\left[\frac{H}{1000}\right]^{0.296}$ $Q_{10} = 85.76A^{0.314}\left[\frac{H}{1000}\right]^{-0.109}$ $Q_{25} = 275.5A^{0.253}\left[\frac{H}{1000}\right]^{-0.555}$ $Q_{50} = 616.9A^{0.281}\left[\frac{H}{1000}\right]^{-0.867}$ $Q_{100} = 1293A^{0.166}\left[\frac{H}{1000}\right]^{-1.154}$

Table 819.7B

Runoff Coefficients for Desert Areas

Type of Drainage Area	Runoff Coefficient
Undisturbed Natural Desert or Desert Landscaping (without impervious weed barrier)	0.30 – 0.40
Desert Landscaping (with impervious weed barrier)	0.55 – 0.85
Desert Hillslopes	0.40 – 0.55
Mountain Terrain (slopes greater than 10%)	0.60 – 0.80

Table 819.7C

Watershed Size for California Desert Regions

Desert Region	Duration (based on Watershed size)
Southern Regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert)	6-hour local storm ($\leq$ 20 mi^2) 6-hour local storm and 24-hour general storm (between 20 mi^2 & 100 mi^2); use the larger peak discharge 24-hour general storm (> 100 mi^2)
Northern Regions (Owens Valley/Mono Lake and Northern Basin and Range)	24-hour general storm

thunderstorm. For these drainage areas, it is recommended that both 6-hour and 24-hour design storm be analyzed, and the storm that produces the largest peak discharge be chosen as the design basis.

Drainage Areas > 100 mi^2 – Since general storms usually cover a larger area and have a longer duration, for drainage areas greater than 100 mi^2, a 24-hour design storm is recommended.

b. Depth-Duration-Frequency Characteristics

In 2011, NOAA published updated precipitation-frequency estimates for all of California including the desert regions, often cited as NOAA Atlas 14. This information is available online, via the Precipitation Frequency Data Server at http://hdsc.nws.noaa.gov/hdsc/pfds/ NOAA Atlas 14 supersedes NOAA's previous effort, NOAA Atlas 2, the 2004 Atlas 14 which covered the Southwestern U.S., and California's Department of Water Resources (DWR) Bulletin No. 195, where their coverages overlap.

NOAA Atlas 14 provides a vast amount of information, which includes:

- Point Estimates
- ESRI shapefiles and ArcInfo ASCII grids
- Color cartographic maps: all possible combination of frequencies (2-year to 1,000-year) and durations (5-miunte to 60-day)
- Associated Federal Geographic Data Committee-compliant metadata

- Data series used in the analysis: annual maximum series and partial duration series
- Temporal distributions of heavy precipitation (6-hour, 12-hour, 24-hour and 96-hour)
- Seasonal exceedance graphs: counts of events that exceed the 1 in 2, 5, 10, 25, 50 and 100 annual exceedance probabilities for the 60-minute, 24-hour, 48-hour and 10-day durations

c. Depth-Area Reduction

Depth-area reduction is the method of applying point rainfall data from one or several gaged stations within a watershed to that entire watershed. NOAA Atlas 14 provides high resolution depth-duration frequency point data which can then be computed with other depth-duration frequency data in that cell to obtain an average depth-duration frequency over a watershed. However, as this data is available as point data, the average calculated depth-duration frequency may not represent an entire watershed. To convert this point data into watershed area, a conversion factor may be applied, of which, two methods are available: applying a reduction factor; or applying depth-area reduction curves.

NOAA is currently working on updating the reduction factors, thus, until then, the depth-area reduction curves are recommended. Two depth-area reduction curves are available: (1) the depth curves in National Weather Service's HYDRO-40 (http://www.nws.noaa.gov/oh/hdsc/PF_related_studies/TechnicalMemorandum_HYDRO40.pdf); and (2) the depth curves in NOAA Atlas 2. The general consensus is that the depth curves from HDRO-40 better represent the desert areas of California, and are recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and the Mojave Desert). For the upper regions (Owens Valley/Mono Lake and Northern Basin and Range), the curves from NOAA Atlas 2 are recommended.

The variables needed to apply depth area reduction curves to a watershed are a storm frequency (i.e., a 100-year storm), storm duration (i.e., a 30-minutes storm), and the area of a watershed. For example, if a 100-year storm with a duration of 60-minutes were to be analyzed over a desert watershed of 25 mi^2, then using Figure 819.7B, the Depth-Area Ratio would be 0.64. This ratio would then be multiplied by the averaged point-rainfall data, which would then result in the rainfall over the entire watershed.

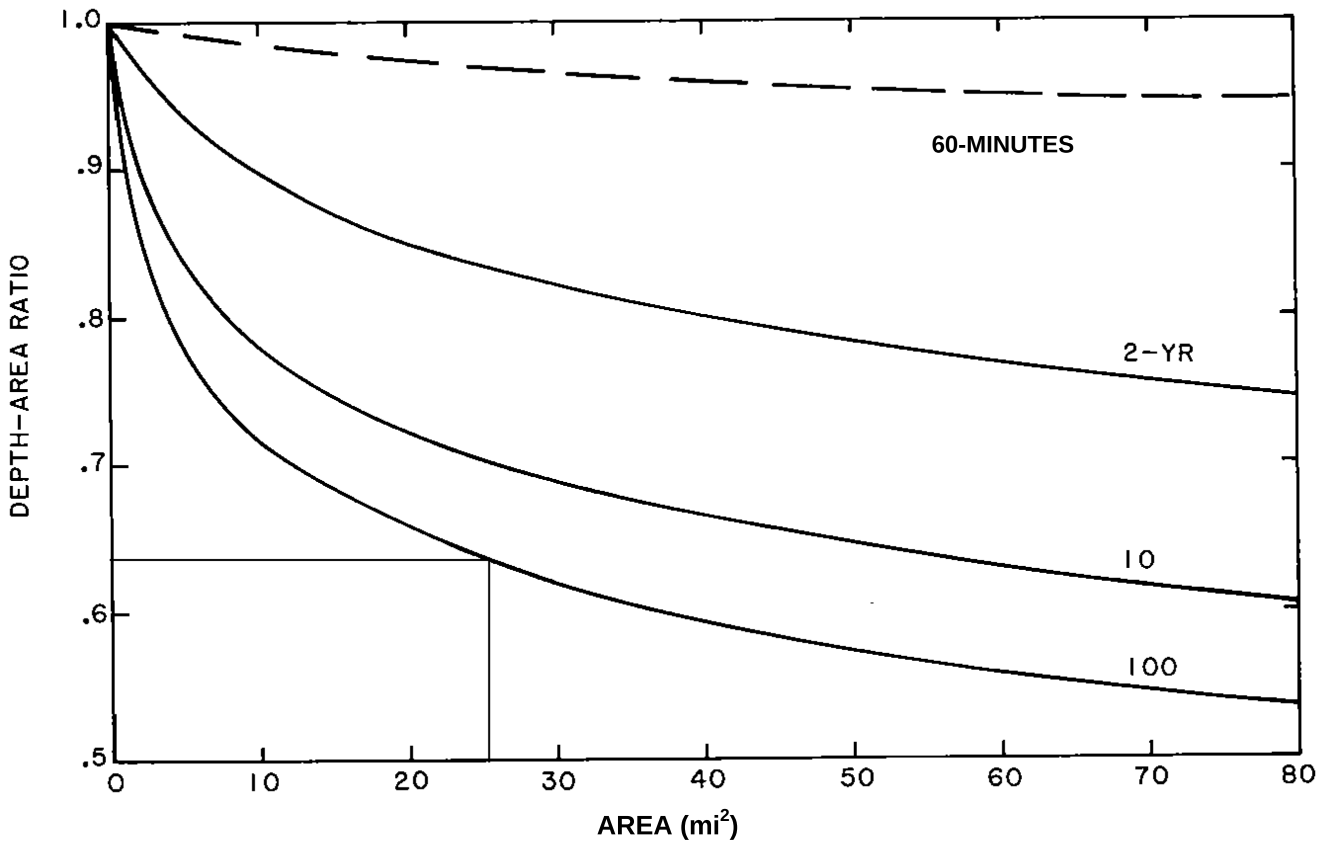
Point rainfall data is available from NOAA Atlas 14, which must then be converted to area rainfall data. Conversions are available in two forms: (1) the National Weather Service's HYDRO-40, and (2) NOAA Atlas 2. The National Weather Service's HYDRO-40 is recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert.) NOAA Atlas 2 is recommended for the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range).

(2) Rainfall Losses

Antecedent Moisture Condition – The Antecedent Moisture Condition (AMC) is the amount of moisture present in the soil before a rainfall event, or

Figure 819.7B

Example Depth-Area Reduction Curve



conversely, the amount of moisture the soil can absorb before becoming saturated (Note: the AMC is also referred to as the Antecedent Runoff Condition [ARC]). Once the soil is saturated, runoff will occur. Generally, the AMC is classified into three levels:

- AMC I – Lowest runoff potential. The watershed soils are dry enough to allow satisfactory grading or cultivation to take place.
- AMC II – Moderate runoff potential. AMC II represents an average study condition.
- AMC III – Highest runoff potential. The watershed is practically saturated from antecedent rainfall.

Because of the different storm types present in California's desert regions, AMC I is recommended as design criteria for local thunderstorms, and AMC II is recommended as design criteria for general storms.

Curve Number – The curve number was developed by the then Soil Conservation Service (SCS), which is now called the National Resource Conservation Service (NRCS). The curve number is a function of land use, soil type and the soil's AMC, and is used to describe a drainage area's storm water runoff potential. The soil type(s) are typically listed by name and can be obtained in the form of a soil survey from the local NRCS office. The soil surveys classify and present the soil types into 4 different hydrological groups, which are shown in Table 819.7D. From the hydrological groups, curve numbers are assigned for each possible land use-soil group combinations, as shown in Table 819.7E. The curve numbers shown in Table 819.7E are representative of AMC II, and need to be converted to represent AMC I, and AMC III, respectively. The following equations to convert an AMC II curve number to an AMC I or AMC III curve number, using a five-day period as the minimum for estimating the AMC's:

$$CN_{AMCI} = \frac{4.2CN_{AMCII}}{10 - 0.058CN_{AMCII}}$$

$$CN_{AMCIII} = \frac{23CN_{AMCII}}{10 + 0.13CN_{AMCII}}$$

Note: The AMC of a storm area may vary during a storm; heavy rain falling on AMC I soil can change the AMC from I to II or III during the storm.

(3) Transformation

Total runoff can be characterized by two types of runoff flow: direct runoff and base flow. Direct runoff is classified as storm runoff occurring during or shortly after a storm event. Base flow is classified as subsurface runoff from prior precipitation events and delayed subsurface runoff from the current storm. The transformation of precipitation runoff to excess can be accomplished using a unit hydrograph approach. The unit hydrograph method is based on the assumption that a watershed, in converting precipitation excess to runoff, acts as a linear, time-invariant system.

Unit Hydrograph Approach

A unit hydrograph for a drainage area is a curve showing the time distribution of runoff that would result at the concentration point from one inch of effective rainfall over the drainage area above that point.

The unit hydrograph method assumes that watershed discharge is related to the total volume of runoff, that the time factors that affect the unit hydrograph shape are invariant, and that watershed rainfall-runoff relationships are characterized by watershed area, slope and shape factors.

Table 819.7D

Hydrologic Soil Groups

Hydrologic Soil Group	Soil Group Characteristics
A	Soils having high infiltration rates, even when thoroughly wetted and consisting chiefly of deep, well to excessively-drained sands or gravels. These soils have a high rate of water transmission.
B	Soils having moderate infiltration rates when thoroughly wetted and consisting of moderately deep to deep, moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

a. SCS Unit Hydrogoraph

The SCS dimensionless unit hydrograph is based on averages of unit hydrographs derived from gaged rainfall and runoff for a large number of small rural basins throughout the U.S. The definition of the SCS unit hydrograph normally only requires one parameter, which is lag, defined as the time from the centroid of precipitation excess to the time of the peak of the unit hydrograph. For ungaged watersheds, the SCS suggests that the unit hydrograph lag time, t_{lag}, may be related to time of concentration t_c, through the following relation:

$$t_{lag} = 0.6t_c$$

The time of concentration is the sum of travel time through sheet flow, shallow concentrated flow, and channel flow segments. A typical SCS Unit Hydrograph is similar to Figure 816.5.

A unit hydrograph can be derived from observed rainfall and runoff, however either may be unavailable. In such cases, a synthetic unit hydrograph can be developed using the S-graph method.

b. S-graph

An S-graph is a summation hydrograph of runoff that would result from the continuous generation of unit storm effective rainfall over the area (1-inch per hour continuously). The S-graph method uses a basic time-runoff relationship for a watershed type in a form suitable for application to ungaged basins, and is based upon percent of ultimate discharge and percent of lag time. Several entities, including local and Federal agencies, have developed location-specific S-Graphs that areapplicable to California's desert regions.

The ordinate is expressed in percent of ultimate discharge, and the abscissa is expressed in percent of lag time. Ultimate discharge, which is the maximum discharge

Table 819.7E

Curve Numbers for Land Use-Soil Combinations

Description	Average % Impervious	Curve Number by Hydrological Soil Group				Typical Land Uses
		A	B	C	D	
Residential (High Density)	65	77	85	90	92	Multi-Family, Apartments, Condos, Trailer Parks
Residential (Medium Density)	30	57	72	81	86	Single-Family, Lot Size ¼ to 1 acre
Residential (Low Density)	15	48	66	78	83	Single-Family, Lot Size 1 acre or greater
Commercial	85	89	92	94	95	Strip Commercial, Shopping Centers, Convenience Stores
Industrial	72	81	88	91	93	Light Industrial, Schools, Prisons, Treatment Plants
Disturbed / Transitional	5	76	85	89	91	Gravel Parking, Quarries, Land Under Development
Agricultural	5	67	77	83	87	Cultivated Land, Row Crops, Broadcast Legumes
Open Land – Good	5	39	61	74	80	Parks, Golf Courses, Greenways, Grazed Pasture
Meadow	5	30	58	71	78	Hay Fields, Tall Grass, Ungrazed Pasture
Woods (Thick Cover)	5	30	55	70	77	Forest Litter and Brush adequately cover soil
Woods (Thin Cover)	5	43	65	76	82	Light Woods, Woods-Grass Combination, Tree Farms
Impervious	95	98	98	98	98	Paved Parking, Shopping Malls, Major Roadways
Water	100	100	100	100	100	Water Bodies, Lakes, Ponds, Wetlands

attainable for a given intensity, occurs when the rate of runoff on the summation hydrograph reaches the rate of effective rainfall.

Lag for a watershed is an empirical expression of the hydrologic characteristics of a watershed in terms of time. It is defined as the elapsed time (in hours) from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50 percent of ultimate discharge. When the lags determined from summation hydrographs for several gaged watersheds are correlated to the hydrologic characteristics of the watersheds, an empirical relationship is usually apparent. This relationship can then be used to determine the lags for comparable ungaged drainage areas for which the hydrologic characteristics can be determined, and a unit hydrograph applicable to the ungaged watersheds can be easily derived.

Figure 819.7C is a sample illustration of a San Bernardino County S-Graph, while Figure 819.7D shows an example S-Graph from USBR.

Recommendations

For watersheds with mountainous terrain/high elevations in the upper portions, the San Bernardino County Mountain S-Graph (http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf) is recommended. For watersheds in the southern desert regions with limited or no mountainous terrain/high elevations, the San Bernardino County Desert S-Graph (http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf) is recommended. The U.S. Bureau of Reclamation (USBR) S-Graph (http://www.usbr.gov/pmts/hydraulics_lab/pubs/manuals/SmallDams.pdf) is recommended for watersheds in the Northern Basin and Range.

As an alternative to the above mentioned S-Graphs, the SCS Unit Hydrograph may also be used.

(4) Channel Routing

Channel routing is a process used to predict the temporal and spatial variation of a flood hydrograph as it moves through a river reach. The effects of storage and flow resistance within a river reach are reflected by changes in hydrograph shape and timing as the flood wave moves from upstream to downstream. The four commonly used methods are the kinematic wave routing, Modified Puls routing, Muskingum routing, and Muskingum-Cunge routing. The advantages and disadvantages for each method are described in Table 819.7F. Table 819.7G provides guidance for selecting an appropriate routing method. The Muskingum-Cunge routing method can handle a wide range of flow conditions with the exception of significant backwater. The Modified Puls routing can model backwater effects. The kinematic wave routing method is often applied in urban areas with well defined channels.

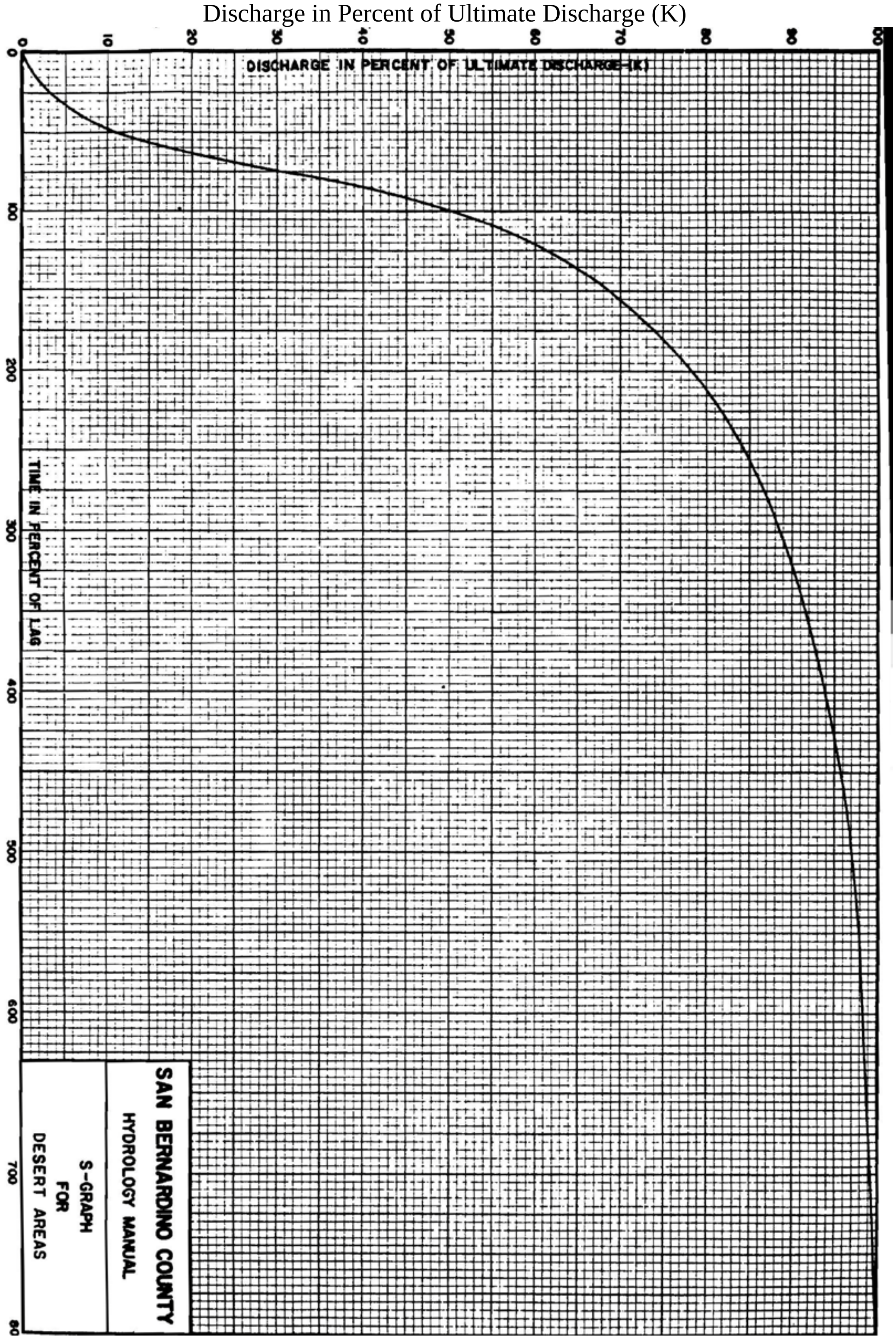
(5) Storm Duration and Temporal Distribution

Temporal distribution is the time-related distribution of the precipitation depth within the duration of the design storm. Temporal distribution patterns of design storms are based on the storm duration. The temporal distribution pattern for short-duration storms represents a single cloudburst and is based on rainfall statistics. The temporal distribution for long-duration storms resembles multiple events and is patterned after historic events.

Figure 819.7C

San Bernardino County Hydrograph for Desert Areas

Discharge in Percent of Ultimate Discharge (K)



Time in Percent of Lag

Figure 819.7D

USBR Example S-Graph

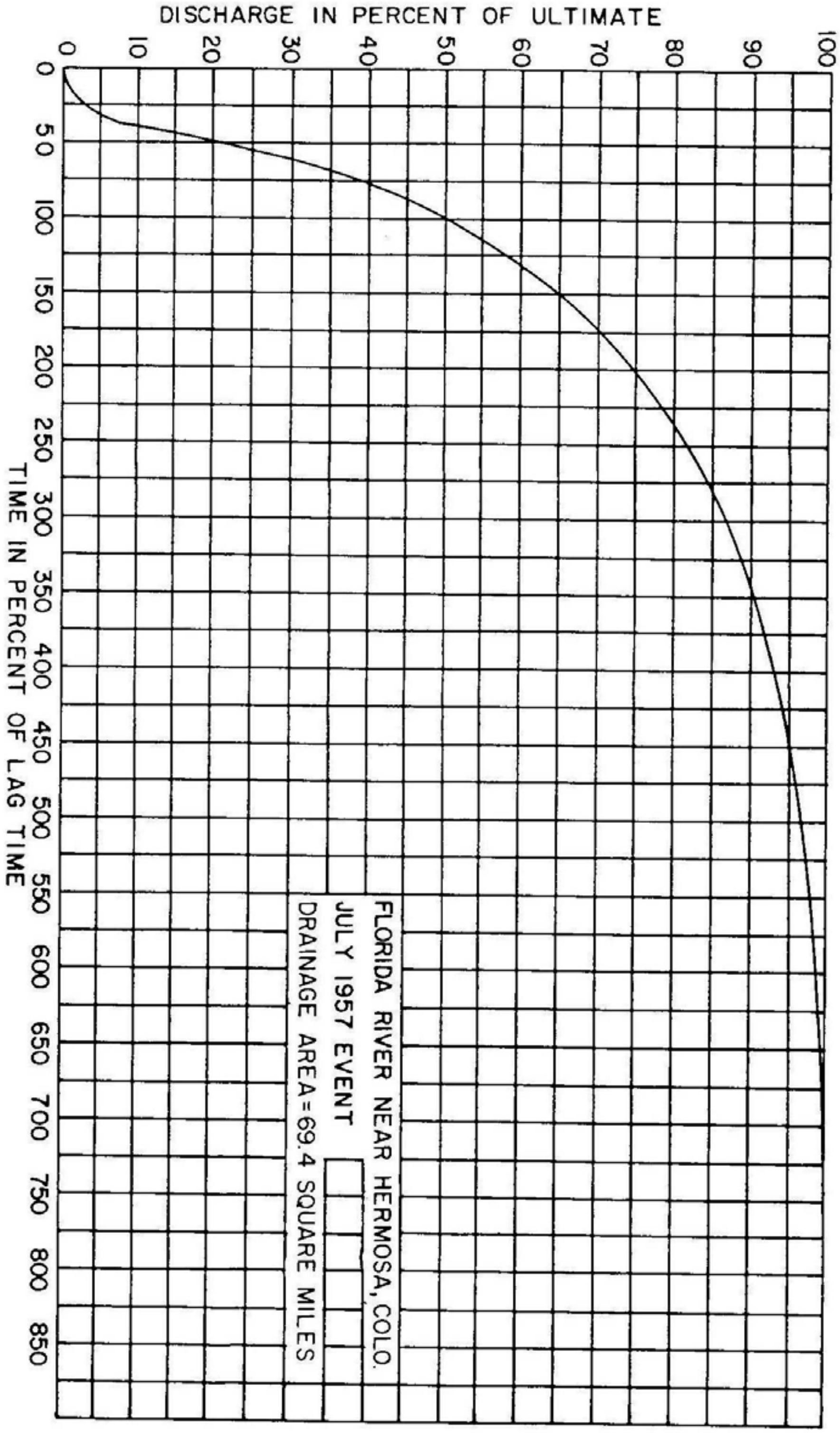


Table 819.7F

Channel Routing Methods

Routing Method	Pros	Cons
Kinetmatic Wave	▪ A conceptual model assuming a uniform flow condition. ▪ In general, works best for steep (10 ft/mile or greater), well defined channels. ▪ It is often applied in urban areas because the routing reaches are generally short and well-defined.	▪ Cannot handle hydrograph attenuation, significant overbank storage, and backwater effects.
Modified Puls	▪ Known as storage routing or level-pool routing. ▪ Can handle backwater effects through the storage-discharge relationship.	▪ Need to use hydraulic model to define the required storage-outflow relationship.
Muskingum	▪ Directly accommodates the looped relationship between storage and outflow. ▪ A linear routing technique that uses coefficients to account for hydrograph timing and diffusion.	▪ The coefficients cannot be used to model a range of floods that may remain in bank or go out of bank. Therefore, not applicable to significant overbank flows.
Muskingum-Cunge	▪ A nonlinear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. ▪ The parameters are physically based. ▪ Has been shown to compare well against the full unsteady flow equations over a wide range of flow conditions.	▪ It cannot account for backwater effects. ▪ Not very applicable for routing a very rapidly rising hydrograph through a flat channel.

Table 819.7G

Channel Method Routing Guidance

If this is true…	… then this routing model may be considered.
No observed hydrograph data available for calibration	Kinematic wave; Muskingum-Cunge
Significant backwater will influence discharge hydrograph	Modified Puls
Flood wave will go out of bank, into floodplain.	Modified Puls; Muskingum-Cunge with 8-point cross section
Channel slope > 0.002 and $\frac{TS_o u_o}{d_o} \geq 171$	Any
Channel slopes from 0.002 to 0.0004 and $\frac{TS_o u_o}{d_o} \geq 171$	Muskingum-Cunge; Modified Puls; Muskingum
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o} \right)^{1/2} \geq 30$	Muskingum-Cunge
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o} \right)^{1/2} < 30$	None

Notes:
- T = hydrograph duration
- u_o = reference mean velocity
- d_o = reference flow depth
- S_o = channel slope

Since the storm events in California's desert regions are made up of two distinct separate storm types, the summer convective storm and the general winter storm, the design storm durations should be adjusted accordingly. For California's desert regions, the 100-year 6-hour storm is recommended for the convective storms, and the 100-year 24-hour storm is recommended for the winter storms. Table 819.7H summarizes the design storm durations for the different desert regions throughout California.

(2) *Sediment/Debris Bulking*

The process of increasing the water volume flow rate to account for high concentrations of sediment and debris is defined as bulking. Debris carried in the flow can be significant and greatly increase flow volume conveyed from a watershed. This condition occurs frequently in mountainous areas subject to wildfires with soil erosion, as well as arid regions around alluvial fans and other geologic activity. By bulking the flow through the use of an appropriate bulking factor, bridge openings and culverts can be properly sized for areas that experience high sediment and debris concentration.

(a) Bulking Factor

Bulking factors are applied to a peak (clear-water) flow to obtain a total or bulked peak flow, which provides a safety factor in the sizing of hydraulic structures. For a given watershed, a bulking factor is typically a function of the historical concentration of sediment in the flow.

(b) Types of Sediment/Water Flow

The behavior of flood flows will vary depending on the concentration of sediment in the mixed flow, where the common flow types are normal stream flow, hyperconcentrated flow, and debris flow.

1. Normal Stream Flow

During normal stream flow, the sediment load minimally influences flow behavior or characteristics. Because sediment has little impact, this type of flow can be analyzed as a Newtonian fluid and standard hydraulic methods can be used. The upper limit of sediment concentration by volume for normal stream flow is 20 percent and bulking factors are applied cautiously because of the low concentration. (See Table 819.7I) The small amount of sediment is conveyed by conventional suspended load and bed-load.

2. Hyperconcentrated Flow

Hyperconcentrated flow is more commonly known as mud flow. Because of potential for large volumes of sand in the water column, fluid properties and transport characteristics change and the mixture does not behave as a Newtonian fluid. However, basic hydraulic methods and models are still generally accepted and used for up to 40 percent sediment concentration by volume. For hyperconcentrated flow, bulking factors vary between 1.43 and 1.67 as shown in Table 819.7I.

3. Debris Flow

In debris flow state, behavior is primarily controlled by the composition of the sediment and debris mixture, where the volume of clay can have a strong influence in the yield strength of the mixture.

During debris flow, which has an upper limit of 50 percent sediment concentration by volume, the sediment/debris/water mixture no longer acts as a Newtonian fluid and basic hydraulic equations do not apply. If detailed hydraulic analysis or modeling of a stream operating under debris flow is needed, FLO2DH is the recommended software choice given its specific debris flow capabilities. HEC-RAS is appropriate for normal stream flow and hyperconcentrated flow, but cannot be applied to debris flow.

For a typical debris flow event, clear-water flow occurs first, followed by a frontal wave of mud and debris. Low frequency events, such as the 100-year flood, most likely contain too much water to produce a debris flow event. Normally, smaller higher frequency events such as 10-year or 25-year floods actually have a greater probability of yielding a debris flow event requiring a higher bulking factor.

As outlined in Table 819.7I, bulking factors for debris flow vary between 1.67 and 2.00.

(c) Sediment/Debris Flow Potential

1. Debris Hazard Areas

Mass movement of rock, debris, and soil is the main source of bulked flows. This can occur in the form of falls, slides, or flows. The volume of sediment and debris from mass movement can enter streams depending upon hydrologic and geologic conditions.

The location of these debris-flow hazards include:

(1) At or near the toe of slope 2:1 or steeper

(2) At or near the intersection of ravines and canyons

(3) Near or within alluvial fans

(4) Soil Slips

Soil slips commonly occur at toes of slope between 2:1 and 3:1. Flowing mud and rocks will accelerate down a slope until the flow path flattens. Once energy loss occurs, rock, mud, and vegetation will be deposited. Debris flow triggered by soil slips can become channelized and travel distances of a mile or more. Figure 819.7E shows the potential of soil slip versus slope angle. As seen in this Figure, the flatter the slope angle, the less effect on flow speed and acceleration.

Table 819.7H

Design Storm Durations

Drainage Area	Desert Region	100-year, 6-hour Convective Storm (AMC I)	100-year, 24-hour General Storm (AMC II)	Regional Regression Equations
> 20 mi^2	Colorado Desert	X		
	Sonoran Desert	X		
	Mojave Desert	X		
	Antelope Valley Desert	X		
< 20 mi^2	Colorado Desert	X*	X*	
	Sonoran Desert	X*	X*	
	Mojave Desert	X*	X*	
	Antelope Valley Desert	X*	X*	
	Owens Valley/Mono Lake			X**
	Northern Basin & Range		X	

* For watersheds greater than 20 mi^2 in the southern desert regions, both the 6-hour Convective Storm (AMC I) and the 24-hour General Storm (AMC II) should be analyzed and the larger of the two peak discharges selected.

** The use of regional regression equations is recommended where streamgage data are not available; otherwise, hydrologic modeling could be performed with snowmelt simulation.

Table 819.7I

Bulking Factors & Types of Sediment Flow

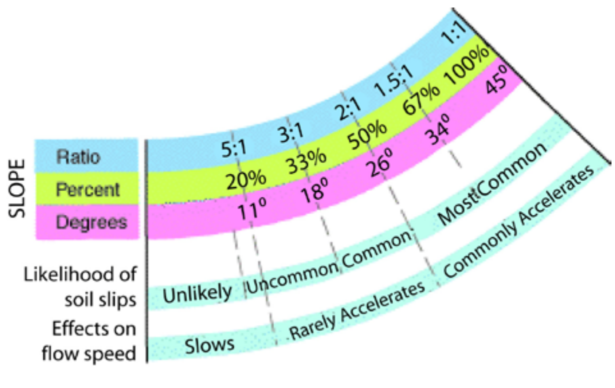
Sediment Flow Type	Bulking Factor	Sediment Concentration by Weight (100% by WT = 1 x 10^6 ppm)	Sediment Concentration by Volume (specific gravity = 2.65)
Normal Streamflow	0	0	0
	1.11	23	10
	1.25	40	20
Hyperconcentrated Flow	1.43	52	30
	1.67	53	40
Debris Flow	2.00	72	50
Landslide	2.50	80	60
	3.33	87	70

2. Geologic Conditions

In the Transverse Ranges that include the San Gabriel and San Bernardino Mountains along the southern and southwestern borders of the Antelope Valley (Region 3) and Mojave Desert (Region 4), their substrate contains sedimentary rocks, fractured basement rocks, and granitic rocks. This type of geology has a high potential of debris flow from the hillsides of these regions.

Figure 819.7E

Soil Slips vs. Slope Angle



While debris flow potential is less prevalent, it is possible to have this condition in the Peninsula Ranges that include the San Jacinto, Santa Rosa, and Laguna Mountains along the western border of the Colorado Desert (Region 1).

(d) Alluvial Fans

An alluvial fan is a landform located at the mouth of a canyon, formed in the shape of a fan, and created over time by deposition of alluvium. With the apex of the fan at the mouth of a canyon, the base of the fan is spread across lower lying plains below the apex. Over time, alluvial fans change and evolve when sediment conveyed by flood flows or debris flows is deposited in active channels, which creates a new channel within the fan. Potentially, alluvial fan flood and debris flows travel at high velocity, where large volumes of sediment can be eroded from mountain canyons down to the lower fan surface. Given this situation, the alignments of the active channels and the overall footprint of an alluvial fan are dynamic. Also, the concentration of sediment/debris volume is dynamic, ranging from negligible to 50 percent.

Alluvial fans can be found on soil maps, geologic maps, topographic maps, and aerial photographs, in addition to the best source which is a site visit. An example of an alluvial fan, shown in plan view, is in Figure 819.7F and Figure 872.3.

Figure 819.7F

Alluvial Fan

(e) Wildfire and Debris Flow

After fires have impacted a watershed, sediment/debris flows are caused by surface erosion from rainfall runoff and landsliding due to rainfall infiltration into the soil. The most dominant cause is the runoff process because fire generally reduces the infiltration and storage capacity of soils, which increases runoff and erosion.

1. Fire Impacts

   Arid regions do not have the same density of trees and vegetation as a forested area, but the arid environment still falls victim to fires in a similar manner. Prior to a fire, the arid region floor can contain a litter layer (leaves, needles, fine twigs, etc.), as well as a duff layer (partially decomposed components of the litter layer). These layers absorb water, provide storage of rainfall, and protect hillsides. Once these layers are burned, they become ash and charcoal particles that seal soil pores and decrease infiltration potential of the soil, which ultimately increases runoff and erosion.

   In order to measure the burn severity of watersheds with respect to hydrologic function, classes of burn severity have been created. These classes are simply stated as high, moderate, low, and unburned. From moderate and high burn severity slopes, the generated sediment can reach channels and streams causing bulked water flows during storm events. Generally speaking, the denser the vegetation in a watershed prior to a fire and the longer a fire burns within this watershed, the greater the effects on soil hydrologic function. This occurs due to the fire creating a water repellent layer at or near the soil surface, the loss of soil structural stability, which all results in more runoff and erosion. After a one or two-year period, the water repellent layer is usually washed away.

(f) Local Agency Methods For Predicting Bulking Factors

1. San Bernardino County

   Instead of conducting a detailed analysis, San Bernardino Flood Control District uses a set value for bulking of 2 (i.e., 100 percent bulking) for any project where bulking flows may be anticipated. This bulking factor of 2 can also be expressed as a 50 percent sediment concentration by volume, which is about the upper limit of debris flow. A higher percentage of sediment concentration would be considered a landslide instead of debris flow. Basically, the San Bernardino County method assumes debris flow conditions for all types of potential bulking.

2. Los Angeles County

   The Los Angeles (LA) County method uses a watershed-specific bulking factor. The LA County Sedimentation Manual, which is located at http://ladpw.org/wrd/publication, divides the county into three basins: LA Basin, Santa Clara River Basin, and Antelope Valley, where only the latter is located in the Caltrans desert hydrology regions. The production of sediment from these basins is dependent upon many factors, including rainfall intensity, vegetative cover, and watershed slope. For each of the LA County basins, Debris Potential Area (DPA) zones have been identified.

   The Design Debris Event (DDE) is associated with the 50-year, 24-hour duration storm, and produces the quantity of sediment from a saturated watershed that is recovered from a burn. For example, a DPA 1 zone sediment rate of 120,000 cubic yards per square mile has been established as the DDE for a 1-square mile drainage area. This sediment rate is recommended for areas of high relief and granitic formation found in the San Gabriel Mountains. In other mountainous areas in LA County, lower sediment rates have been assigned based on differences in topography, geology, and precipitation. For the Antelope Valley basin, eight debris production curves have been generated, and can be found in Appendix B of the LA County Sedimentation Manual along with curves for the other basins.

In addition to sediment production rates, a series of peak bulking factor curves are presented for each LA County basin in Appendix B of the LA manual. The peak bulking factor can be estimated using these curves based on the watershed area and the DPA. Within the Antelope Valley basin, maximum peak bulking factors range from 1.2 in DPA Zone 11 to 2.00 in DPA Zone 1.

3. Riverside County

For Riverside County, a bulking factor is calculated by estimating a sediment/debris yield rate for a specific storm event, and relating it to the largest expected sediment yield of 120,000 cubic yards per square mile for a 1-square mile watershed from the LA County procedure. This sediment rate from LA County is based on the DPA Zone 1 corresponding to the highest expected bulking factor of 2.00.

The bulking factor equation from the Riverside County Hydrology Manual (http://www.floodcontrol.co.riverside.ca.us/downloads/planning/) is as follows:

$$BF = 1 + \frac{D}{120{,}000}$$

BF = Bulking Factor

D = Design Storm Sediment/Debris Production Rate For Study Watershed (cubic yards/square mile)

4. U.S. Army Corps of Engineers- LA District

This method, located at http://www.spl.usace.army.mil/resreg/htdocs/Publications.html, was originally developed to calculate unit sediment/debris yield values for an "n-year" flood event, and applied to the design and analysis of debris catching structures in coastal Southern California watersheds. The LA District method considers frequency of wildfires and flood magnitude in its calculation of unit debris yield. Even though its original application was intended for coastal-draining watersheds, this method can also be used for desert-draining watersheds for the same local mountain ranges.

The LA District method can be applied to watershed areas between 0.1 and 200 mi^2 that have a high proportion of their total area in steep, mountainous topography. This method is best used for watersheds that have received significant antecedent rainfall of at least 2 inches in 48 hours. Given this criteria, the LA District method is more suited for general storms rather than thunderstorms.

As shown below, this method specifies a few equations to estimate unit debris yield dependent upon the areal size of the watershed. These equations were developed by multiple regression analysis using known sediment/debris data.

For watersheds between 3 and 10 mi^2, the following equations can be used:

$$\log Dy = 0.85 \log Q + 0.53 \log RR$$
$$+ 0.04 \log A + 0.22 FF$$

D_y = Unit Debris Yield (cubic yards/square mile)

RR = Relief Ratio (foot/mile), which is the difference in elevation between the highest and lowest points on the longest watercourse divided by the length of the longest watercourse

A = Drainage Area (acres)

FF = Fire Factor

Q = Unit Peak Runoff (cfs/square mile)

In order to account for increase in debris yield due to fire, a non-dimensional fire factor (FF) is a component in the equation above. The FF varies from 3.0 to 6.5, with a higher factor indicating a more recent fire and more debris yield. This factor is 3.0 for desert watersheds because the threat and effects from fire are minimal.

Because the data used to develop the regression equation was taken from the San Gabriel Mountains, an Adjustment and Transposition (A-T) factor needs to be applied to debris yields from the study watersheds. The A-T factor can be determined using Table 819.7J by finding the appropriate subfactor for each of the four groups (Parent Material, Soils, Channel Morphology, and Hillside Morphology) and summing the subfactors. This sum is the total A-T factor, and it must be multiplied by the sediment/debris yield.

Once the sediment/debris yield value has been determined based on the unit yield, a bulking factor can be calculated using a series of equations. The first equation provides a translation of the clear-water discharge to a sediment discharge. This clear-water discharge should be developed using a hydrograph method and a hydrologic modeling program, such as HEC-HMS.

$$Q_S = aQ_W^n$$

Q_S = Sediment Discharge (cfs)

Q_W = 100-Year Clear-Water Discharge (cfs)

a = Bulking Constant

For a majority of sand-bed streams, the value of "n" is between 2 and 3. When n=2, the bulking factor is linearly proportional to the clear-water discharge. As for the coefficient "a", it is determined with the following equation:

$$a = \frac{V_S}{\Delta t \sum Q_W^2}$$

V_S = Total Sediment Volume (cubic feet)

Δt = Computation Time Interval Used In Developing Hydrograph From Hydrologic Model (e.g. HEC-HMS)

Finally, the bulking factor equation is expressed as follows:

$$BF = \frac{Q_W + Q_S}{Q_W} = 1 + aQ_W^{n-1}$$

(g) Recommended Approach For Developing Bulking Factors

A flow chart outlining the recommended bulking factor process is provided in Figure 819.7H, which considers all bulking methods presented in Topic 819.

As shown in Steps 4 and 5 on Figure 819.7H, a bulking factor can be found by:

a. Identifying the type of flow within a watershed and selecting the corresponding bulking factor, or

b. Using one of the agency methods to calculated the bulking factor.

If the type of flow cannot be identified or the project site does not fall within the recommended boundaries from Figure 819.7H, use the LA District Method because it is the most universal given its use of the Adjustment-Transposition factor based on study watershed properties.

Table 819.7J

Adjustment-Transportation Factor Table

	A-T SUBFACTOR				
	0.25	0.20	0.15	0.10	0.05
PARENT MATERIAL	SUBFACTOR GROUP 1				
Folding	Severe		Moderate		Minor
Faulting	Severe		Moderate		Minor
Fracturing	Severe		Moderate		Minor
Weathering	Severe		Moderate		Minor
SOILS	SUBFACTOR GROUP 2				
Soils	Non-cohesive		Partly Cohesive		Highly Cohesive
Soil Profile	Minimal Soil Profile		Some Soil Profile		Well-developed Soil Profile
Soil Cover	Much Bare Soil in Evidence		Some Bare Soil in Evidence		Little Bare Soil in Evidence
Clay Colloids	Few Clay Colloids		Some Clay Colloids		Many Clay Colloids
CHANNEL MORPHOLOGY	SUBFACTOR GROUP 3				
Bedrock Exposures	Few Segments in Bedrock		Some Segments in Bedrock		Many Segments in Bedrock
Bank Erosion	> 30% of Banks Eroding		10 – 30% of Banks Eroding		< 10% of Banks Eroding
Bed and Bank Materials	Non-cohesive Bed and Banks		Partly Cohesive Bed and Banks		Mildly Cohesive Bed and Banks
Vegetation	Poorly Vegetated		Some Vegetation		Much Vegetation
Headcutting	Many Headcuts		Few Headcuts		No Headcutting
HILLSLOPE MORPHOLOGY	SUBFACTOR GROUP 4				
Rills and Gullies	Many and Active		Some Signs		Few Signs
Mass Movement	Many Scars Evident		Few Signs Evident		No Signs Evident
Debris Deposits	Many Eroding Deposits		Some Eroding Deposits		Few Eroding Deposits
The A-T Factor is the sum of the A-T Subfactors from all 4 Subfactor Groups.					

Figure 819.7H

Recommended Bulking Factor Selection Process

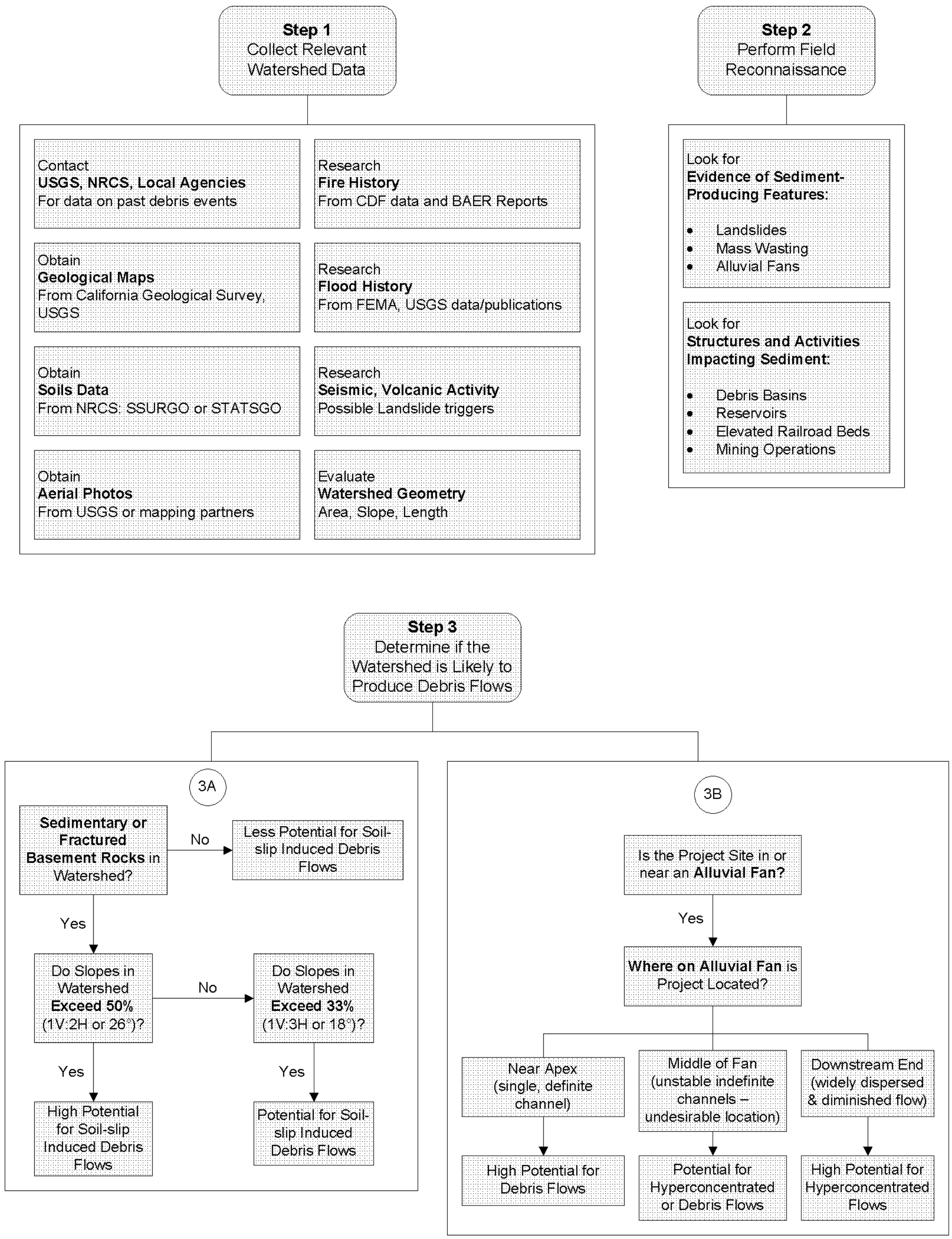
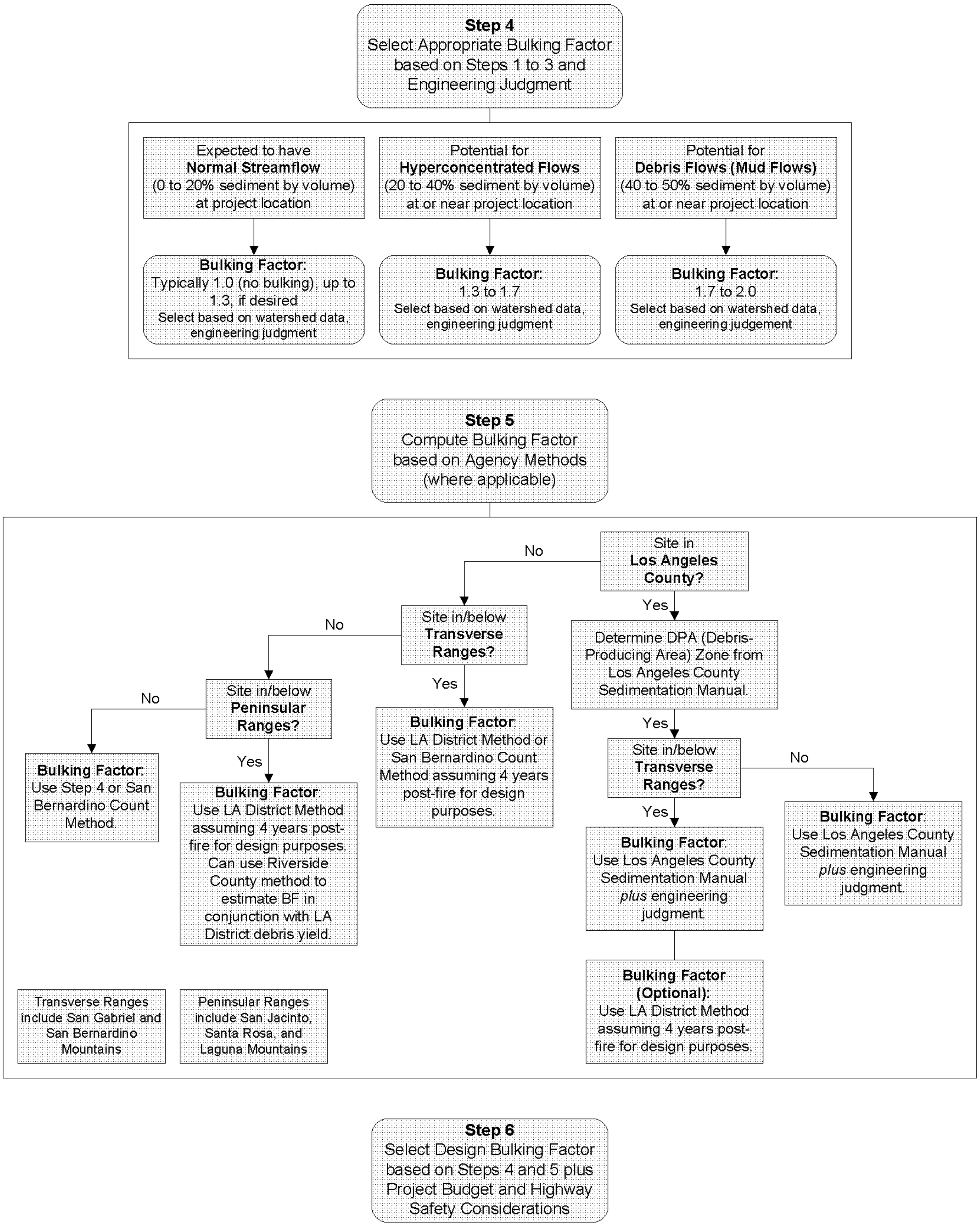


Figure 819.7H

Recommended Bulking Factor Selection Process (Cont'd)



CHAPTER 850
PHYSICAL STANDARDS

Topic 851 - General

Index 851.1 - Introduction

This chapter deals with the selection of drainage facility material type and sizes including pipes, pipe liners, pipe linings, drainage inlets and trench drains.

851.2 Selection of Material and Type

The choice of drainage facility material type and size is based on the following factors:

(1) *Physical and Structural Factors.* Of the many physical and structural considerations, some of the most important are:

(a) Durability.

(b) Headroom.

(c) Earth Loads.

(d) Bedding Conditions.

(e) Conduit Rigidity.

(f) Impact.

(g) Leak Resistance.

(2) *Hydraulic Factors.* Hydraulic considerations involve:

(a) Design Discharge.

(b) Shape, slope and cross sectional area of channel.

(c) Velocity of approach.

(d) Outlet velocity.

(e) Total available head.

(f) Bedload.

(g) Inlet and outlet conditions.

(h) Slope.

(i) Smoothness of conduit.

(j) Length.

Suggested values for Manning's Roughness coefficient (n) for design purposes are given in Table 851.2 for each type of conduit. See Index 864.3 for use of Manning's formula.

Topic 852 - Pipe Materials

852.1 Reinforced Concrete Pipe (RCP)

(1) *Durability.* RCP is generally precast prior to delivery to the project site. The durability of reinforced concrete pipe can be affected by abrasive flows or acids, chlorides and sulfate in the soil and water. See Index 855.2 Abrasion, and Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates.

The following measures increase the durability of reinforced concrete culverts:

(a) Cover Over Reinforcing Steel. Additional cover over the reinforcing steel should be specified where abrasion is likely to be severe as to appreciably shorten the design service life of a concrete culvert. This extra cover is also warranted under exposure to corrosive environments, see Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates. Extra cover over the reinforcing steel does not necessarily require extra wall thickness, as it may be possible to provide the additional cover and still obtain the specified D-load with standard wall thicknesses.

(b) Increase cement content.

(c) Reduce water content.

(d) Invert paving/plating.

(2) *Indirect Design Strength Requirements.*

(a) Design Standards. The “D” load strength of reinforced concrete pipe is determined by the load to produce a 0.01 inch crack under the “3-edge bearing test” called for in AASHTO Designations M 170, M 207M/M 207, and M 206M/M 206 for circular reinforced pipe, oval shaped reinforced pipe, and reinforced concrete pipe arches, respectively.

Table 851.2

Manning "n" Value for Alternative Pipe Materials[1]

Type of Conduit		Recommended Design Value	"n" Value Range
Corrugated Metal Pipe [2]			
(Annular and Helical) [3]			
2⅔" x ½"	corrugation	0.025	0.022 - 0.027
3" x 1"	"	0.028	0.027 - 0.028
5" x 1"	"	0.026	0.025 - 0.026
6" x 2"	"	0.035	0.033 - 0.035
9" x 2½"	"	0.035	0.033 - 0.037
Concrete Pipe			
Pre-cast		0.012	0.011 - 0.017
Cast-in-place		0.013	0.012 - 0.017
Concrete Box		0.013	0.012 - 0.018
Plastic Pipe (HDPE and PVC)			
Smooth Interior		0.012	0.010 - 0.013
Corrugated Interior		0.022	0.020 - 0.025
Spiral Rib Metal Pipe			
¾" (W) x 1" (D) @ 11½" o/c		0.013	0.011 - 0.015
¾" (W) x ¾" (D) @ 7½" o/c		0.013	0.012 - 0.015
¾" (W) x 1" (D) @ 8½" o/c		0.013	0.012 - 0.015
Composite Steel Spiral Rib Pipe		0.012	0.011 - 0.015
Steel Pipe, Ungalvanized		0.015	--
Cast Iron Pipe		0.015	--
Clay Sewer Pipe		0.013	--
Polymer Concrete Grated Line Drain		0.011	0.010 - 0.013

Notes:

(1) Tabulated n-values apply to circular pipes flowing full except for the grated line drain. See Note 5.

(2) For lined corrugated metal pipe, a composite roughness coefficient may be computed using the procedures outlined in the HDS No. 5, Hydraulic Design of Highway Culverts.

(3) Lower n-values may be possible for helical pipe under specific flow conditions (refer to FHWA's publication Hydraulic Flow Resistance Factors for Corrugated Metal Conduits), but in general, it is recommended that the tabulated n-value be used for both annular and helical corrugated pipes.

(4) For culverts operating under inlet control, barrel roughness does not impact the headwater. For culverts operating under outlet control barrel roughness is a significant factor. See Index 825.2 Culvert Flow.

(5) Grated Line Drain details are shown in Standard Plan D98C and described under Index 837.2(6) Grated Line Drains. This type of inlet can be used as an alternative at the locations described under Index 837.2(5) Slotted Drains. The carrying capacity is less than 18-inch slotted (pipe) drains.

(b) Height of Fill. See Topic 856.

(3) *Shapes.* Reinforced concrete culverts are available in circular and oval shapes. Reinforced Concrete Pipe Arch (RCPA) shapes have been discontinued by West Coast manufacturers.

In general, the circular shaped is the most economical for the same cross-sectional area. Oval shapes are appropriate for areas with limited head or overfill or where these shapes are more appropriate for site conditions. A convenient reference of commercially available products and shapes is the AASHTO publication, “A Guide to Standardized Highway Drainage Products”.

(4) *Non-Reinforced Concrete Pipe Option.* Non-reinforced concrete pipe may be substituted at the contractor’s option for reinforced concrete pipe for all sizes 36 inches in diameter and smaller as long as it conforms to Section 65 of the Standard Specifications. Non-Reinforced concrete pipe is not affected by chlorides or stray currents and may be used in lieu of RCP in these environments without coating or the need to provide extra cover over reinforcement.

(5) *Direct Design Method - RCP.* (Contact DES - Structures Design)

852.2 Concrete Box and Arch Culverts

(1) *Box Culverts.* Single and multiple span reinforced concrete box culverts are completely detailed in the Standard Plans. For cast-in-place construction, strength classifications are shown for 10 feet and 20 feet overfills. Precast reinforced concrete box culverts require a minimum of 1 foot of overfill and are not to exceed 12 feet in span length. Special details are necessary if precast boxes are proposed as extensions for existing box culverts. Where the use of precast box culverts is applicable, the project plans should include them as an alternative to cast-in-place construction. Because the standard measurement and payment clauses for precast RCB’s differ from cast-in-place construction, precast units must be identified as an alternative and the special provision must be appropriately modified.

The standard plan sheets for precast boxes show details which require them to be layed out with joints perpendicular to the centerline of the box. This is a consideration for the design engineer in situations which require stage construction and when the culvert is to be aligned on a high skew. This situation will require either a longer culvert than otherwise may have been needed, or a special design allowing for skewed joints. Prior to selecting the latter option DES - Structures Design should be consulted.

(2) *Concrete Arch Culverts.* Technical questions regarding concrete arch culverts should be directed to the Underground Structures Branch of DES - Structures Design.

(3) *Three-Sided Concrete Box Culverts* Design details for cast-in-place (CIP) construction three-sided bottomless concrete box culverts in 2-foot span increments from 12 feet to < 20 feet, inclusive, with strength classifications shown for 10 feet and 20 feet overfills are available upon request from DES - Structures Design. CIP Bottomless Culvert XS-sheets 17-050-1, 2, 3, 4 and 5 may be obtained electronically. Precast three-sided box culverts are an acceptable alternative to CIP designs, where contractors may submit such designs for approval. Both precast and CIP designs must be placed on a foundation designed specifically for the project site.

(4) *Corrosion, Abrasion, and Invert Protection.* Refer to Index 854.2 Abrasion, and Index 854.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates for corrosion, abrasion and invert protection of concrete box and arch culverts.

852.3 Corrugated Steel Pipe, Steel Spiral Rib Pipe and Pipe Arches

Corrugated steel pipe, steel spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes, see Index 852.6. Corrugated steel pipe and pipe arches are available in various corrugation profiles with helical and annular corrugations. Corrugated steel spiral rib pipe is available in several helical corrugation patterns.

(1) *Hydraulics.* Annular and helical corrugated steel pipe configurations are applicable in the situations where velocity reduction is important or if a culvert is being designed with an inlet control condition. Spiral rib pipe, on the other hand, may be more appropriate for use in stormdrain situations or if a culvert is being designed with an outlet control condition. Spiral rib pipe has a lower roughness coefficient (Manning's “n”) than other corrugated metal pipe profiles.

(2) *Durability.* The anticipated maintenance-free service life of corrugated steel pipe, steel spiral rib pipe and pipe arch installations is primarily a function of the corrosivity and abrasiveness of the environment into which the pipe is placed. Corrosion potential must be determined from the pH and minimum resistivity tests covered in California Test 643. Abrasive potential must be estimated from bed material that is present and anticipated flow velocities. Refer to Index 855.1 for a discussion of maintenance-free service life and Index 855.2 Abrasion, and Index 855.3 Corrosion.

The following measures are commonly used to prolong the maintenance-free service life of steel culverts:

(a) Galvanizing. Under most conditions plain galvanizing of steel pipe is all that is needed; however, the presence of corrosive or abrasive elements may require additional protection.

- Protective Coatings - The necessity for any coating should be determined considering hydraulic conditions, local experience, possible environmental impacts, and long-term economy. Approved protective coatings are bituminous asphalt, asphalt mastic and polymeric sheet, which can be applied to the inside and/or outside of the pipe; polymerized asphalt, which is hot-dipped to cover the bottom 90° of the inside and outside of the pipe; and polyethylene for composite steel spiral ribbed pipe which is a steel spiral ribbed pipe externally pre-coated with a polymeric sheet, and internally polyethylene lined. All of these protective coatings are typically shop-applied prior to delivery to the construction site. Polymeric sheet coating provides much improved corrosion resistance over bituminous coatings and can be considered to typically allow achievement of a 50-year maintenance-free service life without need to increase thickness of the steel pipe. To ensure that a damaged coating does not lead to premature catastrophic failure, the base steel thickness for pipes that are to be coated with a polymeric sheet must be able to provide a minimum 10-year service life prior to application of the polymeric material. In addition, a bituminous lining or bituminous paving can be applied over a bituminous coating primer on the inside of the pipe for extra corrosion or abrasion protection (see Section 66 of the Standard Specifications).

Citing Section 5650 of the Fish and Game Code, the Department of Fish and Game (DFG) may restrict the use of bituminous coatings on the interior of pipes if they are to be placed in streams that flow continuously or for an extended period (more than 1 to 2 days) after a rainfall event. Their concern is that abraded particles of asphalt could enter the stream and degrade the fish habitat. Where abrasion is unlikely, DFG concerns should be minimal. DFG has indicated that they have no concerns regarding interior application of polymerized asphalt or polymeric sheet coatings, even under abrasive conditions.

Where the materials report indicates that soil side corrosion is expected, a bituminous asphalt coating which is hot-dipped to cover the entire inside and outside of the pipe or an exterior application of polymeric sheet, as provided in the Standard Specifications, combined with galvanizing of steel, is usually effective in forestalling accelerated corrosion on the backfill side of the pipe. Where soil side corrosion is the only, or primary, factor leading to deterioration, the bituminous asphalt protection layer described above is typically expected to add up to 25 years of service life to an uncoated (i.e., plain galvanized) pipe. A polymeric sheet coating is typically expected to provide up to 50-years of service life to an uncoated pipe. For locations where water side corrosion and/or abrasion is of concern, protective coatings, or protective coatings with pavings, or protective coatings with linings, in combination with galvanizing will add to the culvert service life to a variable degree, depending upon site conditions and type of coating selected. Refer to Index 855.2 Abrasion, and Index 855.3 Corrosion. If hydraulic conditions at the culvert site require a lining on the inside of the pipe or a coating different than that indicated in the Standard Specifications, then the different requirements must be described in the Special Provisions.

- Extra Metal Thickness. Added service life can be achieved by adding metal thickness. However, this should only be considered after protective coatings and pavings have been considered. Since 0.052 inch thick steel culverts is the minimum steel pipe Caltrans allows, it must be limited to locations that are nonabrasive.

  See Table 855.2C for estimating the added service life that can be achieved by coatings and invert paving of steel pipes based upon abrasion resistance characteristics.

(b) Aluminized Steel (Type 2). Evaluations of aluminized steel (type 2) pipe in place for over 40 years have provided data that substantiate a design service life with respect to corrosion resistance equivalent to aluminum pipe. Therefore, for pH values between 5.5 and 8.5, and minimum resistivity values in excess of 1500 ohm-cm, 0.064 inch aluminized steel (type 2) is considered to provide a 50 year design service life. Where abrasion is of concern, aluminized steel (type 2) is considered to be roughly equivalent to galvanized steel. Bituminous coatings are not recommended for corrosion protection, but may be used in accordance with Table 855.2C for abrasion resistance. For pH ranges outside the 5.5 and 8.5 limits or minimum resistivity values below 1500 ohm-cm, aluminized steel (type 2) should not be used. In no case should the thickness of aluminized steel (type 2) be less than the minimum structural requirements for a given diameter of galvanized steel. Refer to Index 855.2 Abrasion, and Index 855.3 Corrosion.

The AltPipe Computer Program is also available to help designers estimate service life for various corrosive/abrasive conditions. See http://www.dot.ca.gov/hq/oppd/altpipe.htm

(3) *Strength Requirements*. The strength requirements for corrugated steel pipes and pipe arches, fabricated under acceptable methods contained in the Standard Specifications, are given in Tables 856.3A, B, C, & D. For steel spiral rib pipe see Tables 856.3E, F & G.

(a) Design Standards.

- Corrugation Profiles - Corrugated steel pipe and pipe arches are available in 2⅔" x ½", 3" x 1", and 5" x 1" profiles with helical corrugations, and 2⅔" x ½" profiles with annular corrugations. Corrugated steel spiral rib pipe is available in a ¾" x ¾" x 7½" or ¾" x 1" x 11½" helical corrugation pattern. For systems requiring large diameter and/or deeper fill capacity a ¾" x 1" x 8½" helical corrugation pattern is available. Composite steel spiral rib pipe is available in a ¾" x ¾" x 7½" helical ribbed profile.

- Metal Thickness - Corrugated steel pipe and pipe arches are available in the thickness as indicated on Tables 856.3A, B, C & D. Corrugated steel spiral rib pipe is available in the thickness as indicated on Tables 856.3E, F & G. Where a maximum overfill is not listed on these tables, the pipe or arch size is not normally available in that thickness. All pipe sections provided in Table 856.3 meet handling and installation flexibility requirements of AASHTO LRFD.

Corrugated steel spiral rib pipe is available in the thickness as indicated on Tables 856.3E, F & G. Composite steel spiral rib pipe is available in the thickness as indicated on Table 856.3G.

- Height of Fill - The allowable overfill heights for corrugated steel and corrugated steel spiral rib pipe and pipe arches for the various diameters or arch sizes and metal thickness are shown on Tables 856.3A, B, C, & D. For corrugated steel spiral rib pipe, overfill heights are shown on Tables 856.3E, F & G. Table 856.3G gives the allowable overfill height for composite steel spiral rib pipe.

(4) *Shapes.* Corrugated steel pipe, steel spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes, see Index 852.6.

(5) *Invert Protection.* Refer to Index 855.2 Abrasion. Invert protection should be considered for corrugated steel culverts exposed to excessive wear from abrasive flows or corrosive water. Severe abrasion usually occurs when the flow velocity exceeds 12 feet per second to 15 feet per second and contains an abrasive bedload of sufficient volume. When severe abrasion or corrosion is anticipated, special designs should be investigated and considered. Typical invert protection includes invert paving with portland cement concrete with wire mesh reinforcement, and invert lining with metal plate. Invert linings should cover the lower fourth of the periphery of circular pipes, and the lower third of pipe arches. Additional metal thickness will increase service life. Reducing the velocity within the culvert is an effective method of preventing severe abrasion. Index 853.6 provides additional guidance on invert paving with concrete.

(6) *Spiral Rib Steel.* Galvanized steel spiral rib pipe is fabricated using sheet steel and continuous helical lock seam fabrication as used for helical corrugated metal pipe. The manufacturing complies with Section 66, “Corrugated Metal Pipe,” of the Standard Specifications, except for profile and fabrication requirements. Spiral rib pipe is fabricated with either: three rectangular ribs spaced midway between seams with ribs 3/4" wide x 3/4" high at a maximum rib pitch of 7-1/2 inches, two rectangular ribs and one half-circle rib equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 11-1/2 inches with the half-circle rib diameter spaced midway between the rectangular ribs, or two rectangular ribs equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 8-1/2 inches.

Aluminized steel spiral rib pipe, type 2 (ASSRP) is available in the same sizes as galvanized steel spiral rib and will support the same fill heights (the aluminizing is simply a replacement coating for zinc galvanizing that allows thinner steel to be placed in certain corrosive environments. See Figure 855.3A for the acceptable pH and resistivity ranges for placement of aluminized steel pipes). Tables 856.3E, F & G give the maximum height of overfill for steel spiral rib pipe constructed under the acceptable methods contained in the Standard Specifications and essentials discussed in Index 829.2.

852.4 Corrugated Aluminum Pipe, Aluminum Spiral Rib Pipe and Pipe Arches

Corrugated aluminum pipe, aluminum spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes see Index 852.6. Corrugated aluminum pipe and pipe arches are available in various corrugation profiles with helical and annular corrugations. Helical corrugated pipe must be specified if anticipated heights of cover exceed the tabulated values for annular corrugated pipe. Non-standard pipe diameters and arch sizes are also available. Aluminum spiral rib pipe is similar to spiral rib steel and is available in several helical corrugation patterns.

(1) *Hydraulics*. Corrugated aluminum pipe comes in various corrugated profiles. Annular and helical corrugated aluminum pipe configurations are applicable in the situations

where velocity reduction is important or if a culvert is being designed with an inlet control condition. Spiral rib pipe, on the other hand, may be more appropriate for use in stormdrain situations or if a culvert is being designed with an outlet control condition. Spiral rib pipe has a lower roughness coefficient (Manning's "n") than other corrugated metal pipe profiles.

(2) *Durability*. Aluminum culverts or stormdrains may be specified as an alternate culvert material. When a 50-year maintenance-free service life of aluminum pipe is required the pH and minimum resistivity, as determined by California Test Method 643, must be known and the following conditions met:

(a) The pH of the soil, backfill, and effluent is within the range of 5.5 and 8.5, inclusive. Bituminous coatings are not recommended for corrosion protection or abrasion resistance. Abrasive potential must be estimated from bed material that is present and anticipated flow velocities. Refer to Index 855.1 for a discussion of maintenance-free service life and Index 855.2 Abrasion, and Index 855.3 Corrosion prior to selecting aluminum as an allowable alternate.

(b) The minimum resistivity of the soil, backfill, and effluent is 1500 ohm-cm or greater.

(c) Aluminum culverts should not be installed in an environment where other aluminum culverts have exhibited significant distress, such as extensive perforation or loss of invert, for whatever reason, apparent or not.

(d) Aluminum may be considered for side drains in environments having the following parameters:

- When pH is between 5.5 and 8.5 and the minimum resistivity is between 500 and 1500 ohm-cm.
- When pH is between 5.0 and 5.5 or between 8.5 and 9.0 and the minimum resistivity is greater than 1500 ohm-cm.

For these conditions, the Corrosion Technology Branch in METS should be contacted to confirm the advisability of using aluminum on specific projects.

(e) Aluminum must not be used as a section or extension of a culvert containing steel sections.

(3) *Strength Requirements.* The strength requirements for corrugated aluminum pipe and pipe arches fabricated under the acceptable methods contained in the Standard Specifications, are given in Tables 856.3H, I & J. See Table 856.3K and Table 856.3L for aluminum spiral rib pipe. Tables 856.3H through L are based on the material properties of H-32 temper aluminum. Additional cover heights can be achieved for an aluminum section when H-34 temper material is used. Contact DES-Structures Design for a special design using H-34 temper material.

(a) Design Standards.

- Corrugation Profiles - Corrugated aluminum pipe and pipe arches are available in 2⅔" x ½" and 5" x 1" profiles with helical or annular corrugations. Aluminum spiral rib pipe is available in a ¾" x ¾" x 7½" or a ¾" x 1" x 11½" helical corrugation profile.
- Metal thickness - Corrugated aluminum pipe and pipe arches are available in the thickness as indicated on Tables 856.3H, I & J. Where a maximum overfill is not listed on these tables, the pipe or pipe arch is not normally available in that thickness. All pipe sections provided in Table 856.3 meet handling and installation flexibility requirements of AASHTO LRFD. Aluminum spiral rib pipe are available in the thickness as indicated on Tables 856.3K & L.
- Height of Fill - The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thicknesses are shown on Tables 856.3H, I & J. For aluminum spiral rib pipe, overfill heights are shown on Tables 856.3K, & L.

(4) *Shapes*. Corrugated aluminum pipe, aluminum spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. Helical corrugated pipe must be specified if anticipated heights of cover exceed the tabulated values for annular corrugated pipe.

For larger diameters, arch spans or special shapes, see Index 852.6. Non-standard pipe diameters and arch sizes are also available.

(5) *Invert Protection.* Invert protection of corrugated aluminum is not recommended.

(6) *Spiral Rib Aluminum.* Aluminum spiral rib pipe is fabricated using sheet aluminum and continuous helical lock seam fabrication as used for helical corrugated metal pipe. The manufacturing complies with Section 66, “Corrugated Metal Pipe,” of the Standard Specifications, except for profile and fabrication requirements. Aluminum spiral rib pipe is fabricated with either: three rectangular ribs spaced midway between seams with ribs 3/4" wide x 3/4" high at a maximum rib pitch of 7-1/2 inches or two rectangular ribs and one half-circle rib equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 11-1/2 inches with the half-circle rib diameter spaced midway between the rectangular ribs. Figure 855.3A should be used to determine the limitations on the use of spiral rib aluminum pipe for the various levels of pH and minimum resistivity.

852.5 Structural Metal Plate

(1) *Pipe and Arches*. Structural plate pipes and arches are available in steel and aluminum for the diameters and thickness as shown on Tables 856.3M, N, O & P.

(2) *Strength Requirements*.

(a) Design Standards.

- Corrugation Profiles - Structural plate pipe and arches are available in a 6" x 2" corrugation for steel and a 9" x 2½" corrugation profile for aluminum.
- Metal Thickness - structural plate pipe and pipe arches are available in thickness as indicated on Tables 856.3M, N, O & P.
- Height of Fill - The allowable height of cover over structural plate pipe and pipe arches for the available diameters and thickness are shown on Tables 856.3M, N, O & P.

Where a maximum overfill is not listed on these tables, the pipe or arch size is not normally available in that thickness. All pipe sections provided in Table 856.3 conform to handling and installation flexibility requirements of AASHTO LRFD. Strutting of culverts, as depicted on Standard Plan D88A, is typically necessary if the pipe is used as a vertical shaft or if the backfill around the pipe is being removed in an unbalanced manner.

(b) Basic Premise. To properly use the above mentioned tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover, and pipe or arch size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

(c) Limitations. In using the tables, the following restrictions should be kept in mind.

- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. For steel pipe or pipe arches, where abrasive conditions are anticipated, additional metal thickness

for the invert plate(s) or a paved invert should be provided when required to fulfill the design service life requirements. Table 855.2C may be used. See Index 855.2 Abrasion and Tables 855.2A, 855.2D and 855.2F.

- Where needed, adequate provisions for corrosion resistance must be made to achieve the required design service life called for in the references mentioned herein.
- Tables 856.3M & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a bearing pressure of 3 tons per square foot at the corners.

(d) Special Designs. If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for structural plate pipe or pipe arches are based, a special design prepared by DES - Structures Design is required.

(3) Arches. Design details with maximum allowable overfills for structural plate arches, with cast in place concrete footings may be obtained from DES - Structures Design.

(4) Vehicular Underpasses. Design details with maximum allowable overfills for structural plate vehicular underpasses with spans from 12 feet 2 inches to 20 feet 4 inches, inclusive, are given in the Standard Plans. These designs are based on "factored" bearing soil pressures from 2.5 tons per square foot to 11 tons per square foot.

(5) Special Shapes.

(a) Long Span.

- Arch
- Low Profile Arch
- High Profile Arch

(b) Ellipse. (Text Later)

- Vertical
- Horizontal

(6) Tunnel Liner Plate. The primary applications for tunnel liner plate include lining large structures in need of a structural repair, or culvert installations through an existing embankment that can be constructed by conventional tunnel methods. Typically, tunnel liner plate is not used for direct burial applications where structural metal plate pipe is recommended. DES - Structures Design will prepare designs upon request. See Index 853.7 for structural repairs.

852.6 Plastic Pipe

Plastic pipe is a generic term which currently includes two independent materials; the Standard Specifications states plastic pipe shall be made of either high density polyethylene (HDPE) or polyvinyl chloride (PVC) material. See Index 852.7(2)(a) Strength Requirements for allowed materials and wall profile types.

(1) Durability. Caltrans standards regarding the durability of plastic pipe are based on the long term performance of its material properties. Both forms of plastic pipe culverts (HDPE and PVC) exhibit good abrasion resistance and are virtually corrosion free. See Index 855.2 Abrasion and Index 855.5 Material Susceptibility to Fire. Also, see Tables 855.2A, 855.2E and 855.2F. The primary environmental factor currently considered in limiting service life of plastic materials is ultraviolet (UV) radiation, typically from sunlight exposure. While virtually all plastic pipes contain some amount of UV protection, the level of protection is not equal. Polyvinyl chloride resins used for pipe rarely incorporate UV protection (typically Titanium Dioxide) in amounts adequate to offset long term exposure to direct sunlight. Therefore, frequent exposure (e.g., cross culverts with exposed ends) can lead to brittleness and such situations should be avoided. Conversely, testing performed to date on HDPE products conforming to specification requirements for inclusion of carbon black have exhibited adequate UV resistance. PVC pipe exposed to freezing conditions can also experience brittleness and such situations should be avoided if there is potential for impact loadings, such as maintenance equipment or

heavy (3" or larger) bedload during periods of freeze. Plastic pipes can also fail from long term stress that leads to crack growth and from chemical degradation. Improvements in plastic resin specifications and testing requirements has led to increased resistance to slow crack growth. Inclusion of anti-oxidants in the material formulation is the most common form of delaying the onset of chemical degradation, but more thorough testing and assessment protocols need to be developed to more accurately estimate long term performance characteristics and durability.

(2) *Strength Requirements.*

(a) Design Standards

- Materials - Plastic pipe shall be either Type C (corrugated exterior and interior) corrugated polyethylene pipe, Type S (corrugated exterior and smooth interior) corrugated polyethylene pipe, or corrugated polyvinyl chloride pipe.
- Height of Fill - The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5.

852.7 Special Purpose Types

(1) *Smooth Steel.* Smooth steel (welded) pipe can be utilized for drainage facilities under conditions where corrugated metal or concrete pipe will not meet the structural or design service life requirements, or for certain jacked pipe operations (e.g., auger boring).

(2) *Composite Steel Spiral Rib Pipe.* Composite steel spiral rib pipe is a smooth interior pipe with efficient hydraulic characteristics. See Table 851.2.

Composite steel spiral rib pipe with its interior polyethylene liner exhibits good abrasion resistance and also resists waterside corrosion found in a typical stormdrain or culvert environment. The exterior of the pipe is protected with a polyethylene film, which offers resistance to corrosive backfills. The pipe will meet a 50 - year maintenance-free service life under most conditions.

(3) *Proprietary Pipe.* See Indexes 110.10 and 601.5(3) for further discussion and guidelines on the use of proprietary items.

Topic 853 - Pipe Liners and Linings for Culvert Rehabilitation

853.1 General

This topic discusses alternative pipe liner and pipe lining materials specifically intended for culvert repair and does not include materials used for Trenchless Excavation Construction (e.g., pipe jacking, pipe ramming, augur boring), joint repair, various types of grouting, or standard pipe materials that are presented elsewhere in Chapter 850 and in the Standard Plans and Standard Specifications.

Many new products and techniques have been developed that often make complete replacement with open cut as shown in the Standard Plans unnecessary. When used appropriately, these new products and techniques can benefit the Department in terms of increased mobility, cost, and safety to both the public and contractors. Design Information Bulletin 83 (DIB 83) outlines a collection of procedures that are cost-effective for their location and that will meet the needs of their particular area and supplements Topic 805. Use the following link; http://www.dot.ca.gov/hq/oppd/dib/dib83-01.htm

853.2 Caltrans Host Pipe Structural Philosophy

In general, if the host (i.e., existing) pipe cannot be made capable of sustaining design loads, it should be replaced rather than rehabilitated. This is a conservative approach and when followed eliminates the need to make a detailed evaluation of the liner's ability to effectively accept and support dead and live loads. Prior to making the decision whether or not to rehabilitate the culvert and/or which method to choose, a determination of the structural integrity of the host pipe must be made. If rehabilitation of the culvert is determined to be a feasible option, existing voids within the culvert backfill or in the base material under the existing culvert identified either by Maintenance (typically as part of their culvert management system) or already noted in the Geotechnical Design Report, should be filled with grout to re-establish its load carrying capability. Therefore, structural considerations for pipe liners are generally limited to their ability to withstand construction handling and/or grouting pressures. When a structural repair is needed, contact Underground Structures within DES – Structures Design. See Index 853.7.

853.3 Problem Identification and Coordination

Before various alternatives for liners or linings can be selected, the first step following a site investigation which may include taking soil and water samples and pipe wall thickness measurements, is to determine the actual cause of the problem. Relative to Caltrans host pipe structural philosophy, the host pipe may be in need of stabilization, rehabilitation or replacement. Further, it will need to be determined if the structure is at the end of its maintenance-free service life, whether it has been damaged by mechanical abrasion, or corrosion (or both) and if there are any changes to the hydrology or habitat (e.g. fish passage). To make these determinations, the Project

Engineer should coordinate with the District Maintenance Culvert Inspection team, Hydraulics and Environmental units. Further assistance may be needed from Geotechnical Design, the Corrosion Technology Branch within DES, Underground Structures and/or Structures Maintenance within DES. Prior to a comprehensive inspection either by trained personnel or camera, it may also be necessary to first clean out the culvert. Problem identification and assessment, and coordination with Headquarters and DES, is discussed in greater detail in DIB 83. Use the following link; http://www.dot.ca.gov/hq/oppd/dib/dib83-01-7.htm#7-1-6

853.4 Alternative Pipe Liner Materials

Similar to the basic policy in Topic 857.1 for alternative pipes, when two or more liner materials meet the design service life and minimum thickness requirements for various materials that are outlined under Topic 855, as well as hydraulic requirements, the plans and specifications should provide for alternative pipe liners to allow for optional selection by the contractor. A table of allowable alternative pipe liner materials for culverts and drainage systems is included as Table 853.1A. This table also identifies the various diameter range limitations and whether annular space grouting is needed. Sliplining consists of sliding a new culvert inside an existing distressed culvert as an alternative to total replacement. See DIB No 83; http://www.dot.ca.gov/hq/oppd/dib/dib83-01-6.htm#6-1-3-1.

The plastic pipeliners listed in the notes under Table 853.1A are installed as slipliners, however, other standard pipe types that are described in Topic 852 (e.g., metal), may be equally viable as material options to be added as sliplining alternatives.

Table 853.1A
Allowable Alternative Pipe Liner Materials

Allowable Alternatives	Diameter Range (1)	Annular Space Grouting
PP (2)	15" – 120"	Yes
CIPP	8" – 96"	No
DRHDPEPL	18" – 30"	No
MSWPVCPLED	6" – 30"	No
SWPVCPLFD	21" – 108"	Yes

Abbreviations:

PP	–	Plastic Pipe (sliplining)
CIPP	–	Cured in Place Pipe
DRHDPEPL	–	Deformed/Reformed HDPE Pipe Liner
SWPVCPLFD	–	Spiral Wound PVC Pipe Liner (Fixed Diameter)
MSWPVCPLED	–	Machine Spiral Wound PVC Pipe Liner (Expandable Diameter)

Note:

(1) Headquarters approval needed for pipe liner diameters 60 inches or larger. Diameter range represents liners only, not Caltrans standard pipe.

(2) At the Contractor's option, plastic pipeliners shall be either:

- Type S or Type C corrugated high density polyethylene (HDPE) pipe conforming to the provisions in Section 64, "Plastic Pipe," of the Standard Specifications; or
- Standard Dimension Ratio (SDR) 35 polyvinyl chloride (PVC) pipe conforming to the requirements

in AASHTO Designation: M 278 and ASTM Designation: F 679; or

- Polyvinyl chloride (PVC) closed profile wall pipe conforming to the requirements in ASTM Designation: F 1803, F 794 (Series 46); or
- Polyvinyl chloride (PVC) dual wall corrugated pipe conforming to the requirements in ASTM Designation: F 794 (Series 46), F 949; or
- High density polyethylene (HDPE) solid wall pipe conforming to the requirements in AASHTO M 326 and ASTM Designation: F 714; or
- Large diameter high density polyethylene (HDPE) closed profile wall pipe conforming to the requirements in ASTM Designation: F 894.

Table 853.1B provides a guide for plastic pipeliner selection in abrasive conditions to achieve a 50-year maintenance-free service life.

For further information on sliplining using plastic pipe liners including available dimensions and stiffness, see DIB 83. Use the following link: http://www.dot.ca.gov/hq/oppd/dib/dib83-01-6.htm#6-1-3-1-1

853.5 Cementitious Pipe Lining

This method may be used to line corroded corrugated steel pipes ranging from 12 inches to a maximum of 48 inches diameter and involves lining an existing culvert with concrete, shotcrete or mortar using a lining machine. Regardless of type of cementitious material used, the resulting lining is a minimum of one inch thick when measured over the top of corrugation crests and has a smooth surface texture. As with other liners, the pipes must first be thoroughly cleaned and dried. For diameters between 12 and 24 inches, the cement mortar is applied by robot. The mortar is pumped to a head, which rotates at high speed using centrifugal force to place the mortar on the walls. A conical-shaped trowel attached to the end of the machine is used to smooth the walls. The maximum recommended length of small-diameter pipe that can be lined using this method is approximately 650 feet. Although this method will line larger diameter pipes, it is mostly appropriate for non-human entry pipes (less than 30 inches). Generally, most problems with steel pipe are limited to the lower 180 degrees, therefore, in larger diameter metal pipes where human entry is possible, invert paving may be all that is required. See Index 853.6.

853.6 Invert Paving with Concrete

(1) *Existing Corrugated Metal Pipe (CMP)*. One of the most effective ways to rehabilitate corroded and severely deteriorated inverts of CMP that are large enough for human entry (with equipment) is by paving them with reinforced concrete using Class 1, Class 2 or shotcrete with a minimum compressive strength of 6000 psi. See index 110.12 Tunnel Safety Orders. Generally, this method is feasible for pipes 48 inches in diameter and larger. If abrasion is present, the aggregate source should be harder material than the streambed load and have a high durability index (consult with District Materials Branch for sampling and recommendation). The maximum grading specified (1.5 inch) for coarse aggregate may need to be modified if the concrete must be pumped. The abrasion resistance of cementitious materials is affected by both its compressive strength and hardness of the aggregate. There is a correlation between decreasing the water/cement ratio, increasing compressive strength and increasing abrasion resistance. Therefore, where abrasion is a significant factor, the lowest practicable water/cement ratios and the hardest available aggregates should be used.

Paving thickness will range from 2 inches to 13 inches depending on abrasiveness of site based on Table 855.2A, and paving limits typically vary from 90 to 120 degrees for the internal angle. See Index 855.2 and Table 855.2F. Note that in Table 855.2F cementitious concrete is not recommended for extremely abrasive conditions (Level 6 in Table 855.2A). For extremely abrasive conditions alternative materials are recommended such as abrasion resistant concrete (calcium aluminate), steel plate or adding RSP. If hydraulically feasible, a flattened invert design may be warranted. Consult the District Hydraulic Branch for a recommendation.

Where there is significant loss of the pipe invert, it may be necessary to tie the concrete to more structurally sound portions of the pipe wall in order to transfer compressive thrust of culvert walls into the invert slab to create a “mechanical” connection using welding studs,

Table 853.1B

Guide for Plastic Pipeliner Selection in Abrasive Conditions[2] to Achieve 50 Years of Maintenance-Free Service Life

		Abrasion Level[1]		
MATERIAL		4	5	6
Type S corrugated polyethylene pipe		-	-	-
Standard Dimension Ratio (SDR) 35 PVC [3]	(46 psi)	4" – 48"	12"- 48"	36"– 48"
	(75 psi)	18" – 48"	18" – 48"	30" – 48"
	(115 psi)	18" – 48"	18" – 48"	27" – 48"
PVC closed profile wall (ASTM F 1803)		18" – 60"	42"– 60"	-
Corrugated PVC (ASTM F 794 & F 949)	(46 psi)	18" – 36"	-	-
	(115 psi)	15"	-	-
Standard Dimension Ratio (SDR) HDPE conforming to: AASHTO M 326 and ASTM Designation F 714	SDR 41	10" – 63"	36" – 63"	-
	SDR 32.5	8" – 63"	30" – 63"	-
	SDR 26	6" – 63"	24" – 63"	-
	SDR 21	5" – 63"	20" – 63"	54" – 63"
	SDR 17	5" – 55"	16" – 55"	42" – 55"
	SDR 15.5	5" – 48"	14" – 48"	42" – 48"
	SDR 13.5	5" – 42"	12" – 42"	34" – 42"
	SDR 11	5" – 36"	10" – 36"	28" - 36"
	SDR 9	5" – 24"	8" – 24"	22"
Polyethylene (PE) large diameter profile wall sewer and drain pipe as specified in ASTM F 894	RSC 160 [4]	18" – 120"	120"	-
	RSC 250 [4]	33" – 108"	96" – 108"	-

Notes:

(1) See Tables 855.2A and 855.2F for Abrasion Level Descriptions and minimum thickness.

(2) No restrictions for Abrasion Levels 1 through 3.

(3) Diameters listed are OD.

(4) RSC = Ring Stiffness Class

angle iron or by other means. When a mechanical connection is used, paving limits may vary up to 180 degrees for the internal angle. These types of repairs should be treated as a special design and consultation with the Headquarters Office of Highway Drainage Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised. Depending on the size of the culvert being paved, pipes with significant invert loss often also have a significant loss of structural backfill with voids present. Where large voids are present, consultation with Geotechnical Services within the Division of Engineering Services (DES) is advised to develop a grouting plan.

See DIB 83 for some invert paving case studies using the following link: http://www.dot.ca.gov/hq/oppd/dib/dib83-01-12.htm#h

(2) Existing RCB and RCP. For existing reinforced concrete boxes (RCB) and reinforced concrete pipes (RCP) with worn inverts and exposed reinforcing steel (generally from abrasive bedloads), the same paving thickness considerations outlined under Index 853.6(1) will apply. However, depending on the structural condition, the existing steel reinforcement may need to be augmented. Consultation with Structures Maintenance and Underground Structures within DES is recommended.

(3) Existing Plastic Pipe. Generally, concrete invert paving is not feasible for plastic pipes because the cement will not adhere to plastic. However, it may be possible to create a "mechanical" connection by other means but these types of repairs should be treated as a special design and consultation with the Headquarters Office of Highway Drainage Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised.

853.7 Structural Repairs with Steel Tunnel Liner Plate

Cracks in RCP greater than 0.1 inch in width and flexible metal pipes with deflections beyond 10 – 12 percent may indicate a serious condition. When replacement is not an option for existing human entry pipes in need of structural repair, an inspection by Structures Maintenance and a structural analysis by Underground Structures within DES are recommended. Further assistance may be needed from Geotechnical Design and/or the Corrosion Unit within DES.

Two flange or four flange steel tunnel liner plate can be specially designed by Underground Structures within DES as a structural repair to accommodate all live and dead loads. The flange plate lap joints facilitate internal bolt connections (structural metal plate requires access to both sides). After the rings have been installed, the annular space between the liner plates and the host pipe is grouted.

Topic 854 - Pipe Connections

854.1 Basic Policy

The Standard Specifications set forth general performance requirements for transverse field joints in all types of culvert and drainage pipe used for highway construction.

Table 857.2 indicates the alternative types of joints that are to be specified for different arch and circular pipe installations with regard to joint strength. The two joint strength types specified for culvert and drainage systems are identified as "standard" and "positive."

(1) Joint Strength. Joint strength is to be designated on the culvert list.

(a) Standard Joints. The "standard" joint is usually for pipes or arches not subject to large soil movement or disjointing forces. These "standard" joints are satisfactory for ordinarily installations, where tongue and groove or simple slip type joints are typically used. The "standard" joint type is generally adequate for underdrains.

(b) Positive Joints. "Positive" joints are for more adverse conditions such as the need to withstand soil movements or resist disjointing forces. Examples of these conditions are steep slopes, sharp curves, and poor foundation conditions. See Index 829.2 for additional discussion. "Positive" joints should always be designated on the culvert list for siphon installations.

(c) Downdrain Joints. Pipe "downdrain" joints are designed to withstand high velocity flows, and to prevent leaking and disjointing that could cause failure.

(d) Joint Strength Properties. A description of the specified joint strength properties tabulated in Section 61 "Culvert and Drainage Pipe Joints" of the Standard Specifications is as follows:

- Shear Strength. The shear strength required of the joint is expressed as a percentage of the calculated shear strength of the pipe at a transverse section remote from the joint. All joints, including any connections must be capable of transferring the required shear across the joint.

- Moment Strength. The moment strength required of the joint is expressed as a percent of the calculated moment capacity of the pipe on a transverse section remote from the joint.

- Tensile Strength. The tensile strength is that which resist the longitudinal force which tends to separate (disjoint) adjacent pipe sections.

- Joint Overlap.

  Integral Preformed Joint. The Joint overlap is the amount of protection of one culvert barrel into the adjacent culvert barrel by the amount specified for the size of pipe designated. The amount of required overlap will vary based on several factors (material type, diameter, etc.) and is designated on the Standard Plans and/or Standard Specifications.

  Any part of an installed joint that has less than ¼ inch overlap will be considered disjointed. Whenever the plans require that the culvert be constructed on a curve, specially manufactured sections of culvert will be required if the design joint cannot meet the minimum ¼ inch overlap requirement after the culvert section is placed on the specified curve.

- Sleeve Joints. The joint overlap is the minimum sleeve width (typically defined by the width of a coupling band) required to engage both the culvert barrels which are abutted to each other.

(2) *Joint Leakage.* The ability of a pipe joint to prevent the passage of either soil particles or water defines its soiltightness or watertightness. These terms are relative and do not mean that a joint will be able to completely stop the movement of soil or water under all conditions. Any pipe joint that allows significant soil migration (piping) will ultimately cause damage to the embankment, the roadway, or the pipe itself. Therefore, site conditions, such as soil particle size, presence of groundwater, potential for pressure flow, etc., must be evaluated to determine the appropriate joint requirement. Other than solvent or fusion welded joints, almost all joints can exhibit some amount of leakage. Joint performance is typically defined by maximum allowable opening size in the joint itself or by the ability to pass a standardized pressure test. The following criteria should be used, with the allowable joint type(s) indicated on the project plans:

- Normal Joint. Many pipe joint systems are not defined as either soiltight or watertight. However, for the majority of applications, such as culverts or storm drains placed in well graded backfill and surrounding soils containing a minimum of fines; no potential for groundwater contact; limited internal pressure, hydraulic grade line below the pavement grade, etc., this type of joint is acceptable. All currently accepted joint types will meet or exceed "Normal Joint" requirements. The following non-gasketed joint types should not be used beyond the "Normal Joint" criteria range:

CMP	-Annular -Hat -Helical -Hugger -2-piece Integral Flange -Universal
PLASTIC	-Split Coupler -Bell/Spigot

- Soiltight Joint. This category includes those joints which would provide an enhanced level of security against leakage and soil migration over the normal joint. One definition of a soiltight joint is contained in Section 26.4.2.4(e) of the AASHTO Standard Specifications for Highway Bridges. In part, this specification requires that if the size of the opening through which soil might migrate exceeds 1/8 inch, the length of the channel (length of path along which the soil particle must travel, i.e., the coupling length) must exceed 4 times the size of the opening. Alternatively, AASHTO allows the joint to pass a hydrostatic test (subjected to approx. 4.6 feet of head) without leaking to be considered soiltight. Typical pipe joints that can meet this criteria are:

RCP and NRCP	-Flared Bell -Flushed Bell -Steel Joint-Flush Bell -Single or Double Offset Design (Flared or Flushed Bell) -Double Gasket -Tounge and Groove* -Self-Centering T & G*
CMP and SSRP	-Annular w/gasket -Hat w/gasket -Helical w/gasket -Hugger w/gasket -2-piece Int. Fl. w/gasket -Universal w/gasket
CSSRP	-Cuffed end w/gasket
PLASTIC	-Split Coupler w/gasket (premium) -Bell/Spigot w/gasket

* Where substantial differential settlement is anticipated, would only meet Normal Joint criteria.

Where soil migration is of concern, but leakage rate is not, a soiltight joint can be achieved in most situations by external wrapping of the joint area with filter fabric (see Index 831.4). Joints listed under both the normal joint and soiltight joint categories, with a filter fabric wrap, would be suitable in these conditions and would not require a gasket or sealant. In many cases, fabric wrapping can be less expensive than a rubber gasket or other joint sealant. Coordination with the District Materials Unit is advised to ensure that the class of filter fabric will withstand construction handling and screen fine soil particles from migrating through the joint.

- Watertight Joint. Watertight joints are specified when the potential for soil erosion or infiltration/exfiltration must be restricted, such as for downdrains, culverts in groundwater zones, etc. Watertight joint requirements are typically met by the use of rubber gasket materials as indicated in the Standard Specifications. The watertight certification test described in Standard Specification Section 61 requires that no leakage occur when a joint is tested for a period of 10 minutes while subjected to a head of 10 feet over the crown of the pipe. This is a test that is typically performed in a laboratory under optimal conditions not typical of those found in the field. Where an assurance of watertightness is needed, a field test should be specified. Designers should be aware that field tests can be relatively expensive, and should only be required if such assurance is critical. A field leakage rate in the range of 700 gallons to 1,000 gallons per inch of nominal diamter per mile of pipe length per day, with a hydrostatic head of 6 feet above the crown of the pipe, is not unusual for joints that pass the watertight certification test, and is sufficiently watertight for well graded, quality backfill conditions. Where conditions are more sensitive, a lower rate should be specified. Rates below 50 to 100 gallons per inch per mile per day are difficult to achieve and would rarely be necessary. For example, sanitary sewers are rarely required to have leakage rates below 200 gallons per inch per mile per day, even though they have stringent health and environmental restrictions. Field hydrostatic tests are typically conducted over a period of 24 hours or more to establish a valid leakage rate. Designers should also be aware that non-circular pipe shapes (CMP pipe arches, RCP oval shapes, etc.) should not be considered watertight even with the use of rubber gaskets or other sealants due to the lack of uniform compression around the periphery of the joint. Additionally, watertight joints specified for pressure pipe or siphon applications must meet the requirements indicated in Standard Specification Sections 65 and 66. Pipe joints that meet Standard Specification Section 61 water-tightness performance criteria are:

RCP and NRCP	-Flared Bell -Flushed Bell -Steel Joint-Flush Bell -Single or Double Offset Design (Flared or Flushed Bell) -Double Gasket
CMP and SSRP	-Hugger Bands (H-10, 12) w/gasket and double bolt bar -Annular Band w/gasket -Two Piece Integral Flange w/sleeve-type gasket*
PLASTIC	-Bell/Spigot w/gasket

* Acceptable as a watertight pipe only in downdrain applications and in 6, 8 and 10 inch diameters. Factory applied sleeve-type gaskets are to be used instead of O-ring or other sealants.

Table 854.1 provides information to help the designer select the proper joint under most conditions.

Topic 855 - Design Service Life

855.1 Basic Concepts

The prediction of design service life of drainage facilities is difficult because of the large number of variables, continuing changes in materials, wide range of environments, and use of various protective coatings. The design service life of a drainage facility is defined as the expected maintenance-free service period of each installation. After this period, it is anticipated major will be needed for the facility to perform as originally designed for further periods.

For all metal pipes and arches that are listed in Table 857.2, maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of perforation at any location on the culvert (See Figures 855.3A, 855.3B, and Tables 855.2D and 855.2F). AltPipe can be used to estimate service life of all circular metal pipe. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe.

For reinforced concrete pipe (RCP), box (RCB) and arch (RCA) culverts, maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of exposed reinforcement at any point on the culvert. AltPipe can be used to estimate service life of reinforced concrete pipe (RCP), but not RCB, RCA or NRCP. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe.

For non-reinforced concrete pipe culverts (NRCP), maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of perforation or major cracking with soil loss at any point on the culvert.

Table 854.1
Joint Leakage Selection Criteria

JOINT TYPE ⇒ ⇓ SITE CONDITIONS	"NORMAL" JOINT	"SOIL TIGHT" JOINT	"WATER TIGHT" JOINT
SOIL FACTORS			
Limited potential for soil migration (e.g., gravel, medium to coarse sands, cohesive soil)	X	X	X
Moderate potential for soil migration (e.g., fine sands, silts)	X(1)	X	X
High potential for soil migration (e.g., very fine sands, silts of limited cohesion)		X(1)	X(1)
INFILTRATION / EXFILTRATION			
No concern over either infiltration or exfiltration.	X	X	X
Infiltration or exfiltration not permitted (e.g., potential to contaminate groundwater, contaminated plume could infiltrate)			X(2)
HYDROSTATIC POTENTIAL			
Installation will rarely flow full. No contact with groundwater.	X	X	X
Installation will occasionally flow full. Internal head no more than 10 feet over crown. No potential groundwater contact.		X	X
Installation may or may not flow full. Internal head no more than 10 feet over crown. May contact groundwater.			X
Possible hydrostatic head (internal or external) greater than 10 feet, but less than 25 ft(3).			X(2)

Notes:

"X" indicates that joint type is acceptable in this application. The designer should specify the most cost-effective option.

(1) Designer should specify filter fabric wrap at joint. See Index 831.4.
(2) Designer should consider specifying field watertightness test.
(3) Pipe subjected to hydrostatic heads greater than 25 ft should have joints designed specifically for pressure applications.

For plastic pipe, maintenance-free service period, with respect to corrosion, abrasion, and long term structural performance, is the number of years from installation until the deterioration reaches the point of perforation at any location on the culvert or until the pipe material has lost structural load carrying capacity typically represented by wall buckling or excessive deflection/deformation. AltPipe can be used to estimate service life of all plastic pipe. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe. All types of culverts are subject to deterioration from corrosion, or abrasion, or material degradation.

Corrosion may result from active elements in the soil, water and/or atmosphere. Abrasion is a result of mechanical wear and depends upon the frequency, duration and velocity of flow, and the amount and character of bedload. Material degradation may result from material quality, UV exposure, or long term material structural performance.

To assure that the maintenance-free service period is achieved, alternative metal pipe may require added thickness and/or protective coatings. Concrete pipe may require extra thickness of concrete cover over the steel reinforcement, high density concrete, using supplementary cementitious materials, epoxy coated reinforcing steel, and/or protective coatings. Means for estimating the maintenance-free service life of pipe, and techniques for extending the useful life of pipe materials are discussed in more detail in Topic 852.

The design service life for drainage facilities for all projects should be as follows:

(1) *Culverts, Drainage Systems, and Side Drains.*

   (a) Roadbed widths greater than 28 feet - 50 years.

   (b) Greater than 10 feet of cover - 50 years.

   (c) Roadbed widths 28 feet or less and with less than 10 feet of cover - 25 years.

   (d) Installations under interim alignment - 25 years.

(2) *Overside Drains.*

   (a) Buried more than 3 feet- 50 years.

   (b) All other conditions, such as on the surface of fill slopes - 25 years.

(3) *Subsurface Drains.*

   (a) Underdrains within roadbed - 50 years.

   (b) Underdrains outside of roadbed - 25 years.

   (c) Stabilization trench drains - 50 years.

In case of conflict in the design service life requirements between the above controls, the highest design service life is required except for those cases of interim alignment with more than 10 feet of cover. For temporary construction, a lesser design service life than that shown above is acceptable.

Where the above indicates a minimum design service life of 25 years, 50 years may be used. For example an anticipated change in traffic conditions or when the highway is considered to be on permanent alignment may warrant the higher design service life.

855.2 Abrasion

All types of pipe material are subject to abrasion and can experience structural failure around the pipe invert if not adequately protected. Abrasion is the wearing away of pipe material by water carrying sands, gravels and rocks (bed load) and is dependent upon size, shape, hardness and volume of bed load in conjunction with volume, velocity, duration and frequency of stream flow in the culvert. For example, at independent sites with a similar velocity range, bedloads consisting of small and round particles will have a lower abrasion potential than those with large and angular particles such as shattered or crushed rocks. Given different sites with similar flow velocities and particle size, studies have shown the angularity and/or volume of the material may have a significant impact to the abrasion potential of the site. Likewise, two sites with similar site characteristics, but different hydrologic characteristics, i.e., volume, duration and frequency of stream flow in the culvert, will probably also have different abrasion levels.

In Table 855.2A six abrasion levels have been defined to assist the designer in quantifying the abrasion potential of a site. The designer is encouraged to use the guidelines provided in Table 855.2A in conjunction with Table 855.2B "Bed

Materials Moved by Various Flow Depths and Velocities" and the abrasion history of a site (if available) to achieve the required service life for a pipe, coating or invert lining material. Sampling of the streambed materials generally is not necessary, but visual examination and documentation of the size and shape of the materials in the streambed and estimating the average stream slope will provide the designer data needed to determine the expected level of abrasion. Where an existing culvert is in place, the condition of the invert and estimated combined wear rate due to abrasion and corrosion based on remaining pipe thickness measurements or if it is known approximately when first perforation occurred (steel pipe only), should always be used first. Figure 855.3B should be used to estimate the expected loss due to corrosion for steel pipe.

The descriptions of abrasion levels in Table 855.2A are intended to serve as general guidance only, and not all of the criteria listed for a particular abrasion level need to be present to justify defining a site at that level. For example, if there are increased velocities with minor bedload volumes, regardless of the gradation, significantly higher velocities may be applicable to any of the lower three abrasion levels and their consideration for use in lieu of one of the upper three abrasion levels is encouraged.

Table 855.2C constitutes a guide for estimating the added service life that can be achieved by coatings and invert paving of steel pipes based upon abrasion resistance characteristics. However, the table does not quantify added service life of coatings and paving of steel pipe based upon corrosion protection. In heavily abrasive situations, concrete inverts or other lining alternatives outlined in Table 855.2A should be considered. The guide values for years of added service life should be modified where field observations of existing installations show that other values are more accurate. The designer should be aware of the following limitations when using Table 855.2C:

- Channel Materials: If there is no existing culvert, it may be assumed that the channel is potentially abrasive to culvert if sand and/or rocks are present. Presence of silt, clay or heavy vegetation may indicate a non-abrasive flow.
- Flow velocities: The velocities indicated in the table should be compared to those generated by the 2-5 year return frequency flood.
- The abrasion levels represent all six abrasion levels presented in Table 855.2A however, levels 2 and 3 have been combined.

Table 855.2D constitutes a guide for anticipated wear (in mils/year) to metal pipe by abrasive channel materials. No additional abrasion wear is anticipated for steel for the lower three abrasion levels defined in Table 855.2A, because it is assumed that there is some degree of abrasion incorporated within California Test 643 and Figure 855.3B. Figure 855.3B, "Chart for Estimating Years to Perforation of Steel Culverts," is part of a Standard California Department of Transportation Test Method derived from highway culvert investigations. This chart alone is not used for determining service life because it does not consider the effects of abrasion or overfill; it is for estimating the years to the first corrosion perforation of the wall or invert of the CSP. Additional gauge thickness or invert protection may be needed if the thickness for structural requirements (i.e., for overfill) is inadequate for abrasion potential.

Table 855.2E indicates relative abrasion resistance properties of pipe and lining materials and summarizes the findings from "Evaluations of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA /CA/TL-CA01-0173 (2007)". This report may be viewed at the following web address: http://www.dot.ca.gov/new/tech/researchreports/reports/2007/evaluation_of_abrasion_resistance_final_report.pdf. See Figure 855.1.

Table 855.2F is based on Tables 855.2D and 855.2E and constitutes a guide for selecting the minimum material thickness of abrasive resistant invert protection for various materials to achieve 50 years of maintenance-free service life.

Structural metal plate pipe and arches provide a viable option for large diameter pipes (60 inches or larger) in abrasive environments because increased thickness can be specified for the lower 180 degrees or invert plates. If the thickness for structural requirements is inadequate for abrasion potential, it is recommended to apply the increased thickness to the lower 180 degrees of the pipe only. Arches, which have a relatively larger invert area than circular pipe, generally will provide a lower abrasion potential from bedload being less concentrated.

Figure 855.1
Abrasion Test Panels

Various culvert material test panels shown in Figure 855.1 after 1 year of wear at site with moderate to severe abrasion (velocities generally exceed 13 ft/s with heavy bedload).

Under similar conditions, aluminum culverts will abrade between one and a half to three times faster than steel culverts. Therefore, aluminum culverts are not recommended where abrasive materials are present, and where flow velocities would encourage abrasion to occur. Culvert flow velocities that frequently exceed 5 feet per second where abrasive materials are present should be carefully evaluated prior to selecting aluminum as an allowable alternate. In a corrosive environment, Aluminum may display less abrasive wear than steel depending on the volume, velocity, size, shape, hardness and rock impact energy of the bed load. However, if it is deemed necessary to place aluminum pipe in abrasion levels 4 through 6 in Table 855.2C, contact Headquarters Office of State Highway Drainage Design for assistance. Invert protection (including concrete) for corrugated aluminum is not recommended.

Aluminized Steel (Type 2) can be considered equivalent to galvanized steel for abrasion resistance and therefore does not have the same limitations as aluminum in abrasive environments.

Concrete pipes typically counter abrasion through increased minimum thickness over the steel reinforcement, i.e., by adding additional sacrificial material. See Table 855.2F. However, there are significantly less limitations involved in increasing the invert thickness of RCB in the field verses increasing minimum thickness over the steel reinforcement of RCP in the plant. Therefore, RCP is typically not recommended in abrasive flows greater than 10 feet per second but may be considered for higher velocities if the bedload is insignificant (e.g. storm drain systems and most culverts smaller than 30 inches in diameter or abrasion levels 1 through 3 in Table 855.2C). Abrasion resistance for any concrete lining is dependent upon the thickness, quality, strength, and hardness of the aggregate and density of the concrete as well as the velocity of the water flow coupled with abrasive sediment content and acidity. Abrasion resistant concrete made from calcium aluminate provides much improved abrasion resistance over cementitious concrete and should be considered as a viable countermeasure in extremely abrasive conditions (i.e, velocity greater than 15 feet per second with heavy bedload). See Table 855.2F.

Plastic materials typically exhibit good abrasion resistance but service life is constrained by the manufactured thickness of typical pipe profiles. Both PVC and HDPE corrugated and ribbed pipe are limited for their use in moderate and heavy bedload abrasion conditions by the combined manufactured inner liner and corrugated wall thicknesses. For culvert rehabilitation, PVC and HDPE pipe slip lining products (e.g. solid wall HDPE) are viable options for applications in moderate and heavy bedload abrasion conditions (see Table 855.2A).

Table 855.2A can be used as a “preliminary estimator” of abrasion potential for material selection to achieve the required service life, however, it uses only two of the factors that are outlined above; bed load size and flow velocity. As discussed above, the other factors that are not used in the table should also be carefully considered. For example, under similar hydraulic conditions, heavy volumes of hard, angular sand may be more abrasive than small volumes of relatively soft, large rocks. Furthermore, two sites with similar site characteristics, but different hydrologic

Table 855.2A
Abrasion Levels and Materials

Abrasion Level	General Site Characteristics	Invert/Pipe Materials
Level 1	• Virtually no bed load with velocities less than 5 ft/s* * Where there are increased velocities with no bed load (e.g. urban storm drain systems or culverts $\leq$ 30" dia.), significantly higher velocities may be applicable to level 1	All pipe materials listed in Table 857.2 allowable for this level. No abrasive resistant protective coatings listed in Table 855.2C needed for metal pipe.
Level 2	• Bed loads of sand, silts, or clays regardless of volume • Velocities $\geq$ 3 ft/s and $\leq$ 8 ft/s** ** Where there are increased velocities with minor bed load volumes (e.g. urban storm drain systems or culverts $\leq$ 30" dia.), significantly higher velocities may be applicable to level 2	All allowable pipe materials listed in Table 857.2 with the following considerations: • Generally, no abrasive resistant protective coatings needed for steel pipe. • Polymeric, polymerized asphalt or bituminous coating or an additional gauge thickness of metal pipe may be specified if existing pipes in the same vicinity have demonstrated susceptibility to abrasion and thickness for structural requirements is inadequate for abrasion potential.
Level 3	• Moderate bed load volumes of sands and gravels (1.5" max). • Velocities > 5 ft/s and $\leq$ 8 ft/s*** *** Where there are increased velocities with <u>minor</u> bed load volumes $\leq$ 1.5" (e.g. storm drain systems or culverts $\leq$ 30" dia.), higher velocities may be applicable to level 3	All allowable pipe materials listed in Table 857.2 with the following considerations: • Steel pipe may need one of the abrasive resistant protective coatings listed in Table 855.2C or additional gauge thickness if existing pipes in the same vicinity have demonstrated susceptibility to abrasion and thickness for structural requirements is inadequate for abrasion potential. • Aluminum pipe may require additional gauge thickness for abrasion if thickness for structural requirements is inadequate for abrasion potential. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (equivalent to galv. Steel) where pH < 6.5 and resistivity < 20,000. Lining alternatives: • PVC, • Corrugated or Solid Wall HDPE, • CIPP

Table 855.2A
Abrasion Levels and Materials (Con't)

Level 4	• Small to moderate bed load volumes of sands, gravels, and/or small cobbles/rocks with maximum stone sizes up to about 6 in. • Velocities > 8 ft/s and ≤ 12 ft/s	All allowable pipe materials listed in Table 857.2 with the following considerations: • Steel pipe will typically need one of the abrasive resistant protective coatings listed in Table 855.2C or may need additional gauge thickness if thickness for structural requirements is inadequate for abrasion potential. • Aluminum pipe not recommended. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 6.5 and resistivity < 20,000 if thickness for structural requirements is inadequate for abrasion potential. • Increase concrete cover over reinforcing steel for RCB (invert only). RCP generally not recommended. • Corrugated HDPE (Type S) limited to ≥ 48" min. diameter. Corrugated HDPE Type C not recommended. • Corrugated PVC limited to ≥ 18" min. diameter Lining alternatives: • Closed profile or SDR 35 PVC (corrugated and ribbed PVC limited to ≥ 18" min. diameter. Machine-wound PVC not recommended. • SDR HDPE (corrugated HDPE Type S and corrugated HDPE Type C not recommended). • CIPP (min. thickness for abrasion specified) • Concrete.

Table 855.2A
Abrasion Levels and Materials (Con't)

Level 5	• Moderate bed load volumes of sands, gravels, and/or small cobbles with maximum stone sizes up to about 6 in. For larger stone sizes within this velocity range, see Level 6 • Velocities > 12 ft/s and $\leq$ 15 ft/s	• Aluminum pipe not recommended. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 6.5 and resistivity < 20,000 if thickness for structural requirements is inadequate for abrasion potential. • Closed profile and SDR 35 PVC liners are allowed but not recommended for upper range of stone sizes in bed load if freezing conditions are often encountered, otherwise allowed for stone sizes up to 3 inches. • Most abrasive resistant coatings listed in Table 855.2C are not recommended for steel pipe. A concrete invert lining or additional gauge thickness is recommended if thickness for structural requirements is inadequate for abrasion potential. See lining alternatives below. • Increase concrete cover over reinforcing steel for RCB (invert only). RCP generally not recommended Lining alternatives: • Closed profile ($\geq$ 42 in) or SDR 35 PVC (corrugated not recommended. Machine-wound PVC not recommended) • SDR HDPE (corrugated Type S and Type C not recommended) • CIPP (with min. thickness for abrasion specified) • Concrete

Table 855.2A
Abrasion Levels and Materials (Con't)

Level 6	• Heavy bed load volumes of sands, gravel and rocks, with stone sizes 6 in or larger • Velocities > 12 ft/s and ≤ 20 ft/s or • Heavy bed load volumes of sands, gravel and small cobbles, with stone sizes up to about 6 in • Velocities > 15 ft/s and ≤ 20 ft/s**** ****Very limited data on abrasion resistance for velocities > 20 ft/s; contact District Hydraulics Branch.	• Aluminum pipe not recommended. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 5.5 and resistivity < 20,000. • None of the abrasive resistant protective coatings listed in Table 855.2C are recommended for protecting steel pipe. • A concrete invert lining and additional gauge thickness is recommended. See lining alternatives below. • Corrugated HDPE not recommended. Corrugated and closed profile PVC pipe not recommended. • RCP not recommended. Increase concrete cover over reinforcing steel recommended for RCB (invert only) for velocities up to 15 ft/s. RCB not recommended for bed load stone sizes > 3 in and velocities greater than 15 ft/s unless concrete lining with larger, harder aggregate is placed (see lining alternatives below). • SDR 35 PVC liners (≥ 27 in) allowed but not recommended for upper range of stone sizes in bed load if freezing conditions are often encountered, otherwise allowed for stone sizes up to 3 in. Lining/replacement alternatives: • SDR 35 PVC (see note above) or HDPE SDR (minimum wall thickness 2.5") • CIPP (with min. thickness for abrasion specified), • Class 2 concrete with embedded aggregate (e.g. cobbles or RSP (facing)): (for all bed load sizes a larger, harder aggregate than the bed load, decreased water cement ratio and an increased concrete compressive strength should be specified). • Alternative invert linings may include steel plate, rails or concreted RSP, and abrasion resistant concrete (Calcium Aluminate). • For new/replacement construction, consider "bottomless" structures.

Table 855.2B
Bed Materials Moved by Various Flow Depths and Velocities

Bed Material	Grain Dimensions (inches)	Approximate Nonscour Velocities (feet per second)			
		Mean Depth (feet)			
		1.3	3.3	6.6	9.8
Boulders	more than 10	15.1	16.7	19.0	20.3
Large cobbles	10 – 5	11.8	13.4	15.4	16.4
Small cobbles	5 – 2.5	7.5	8.9	10.2	11.2
Very coarse gravel	2.5 – 1.25	5.2	6.2	7.2	8.2
Coarse gravel	1.25 – 0.63	4.1	4.7	5.4	6.1
Medium gravel	0.63 – 0.31	3.3	3.7	4.1	4.6
Fine gravel	0.31 – 0.16	2.6	3.0	3.3	3.8
Very fine gravel	0.16 – 0.079	2.2	2.5	2.8	3.1
Very coarse sand	0.079 – 0.039	1.8	2.1	2.4	2.7
Coarse sand	0.039 – 0.020	1.5	1.8	2.1	2.3
Medium sand	0.020 – 0.010	1.2	1.5	1.8	2.0
Fine sand	0.010 – 0.005	0.98	1.3	1.6	1.8
Compact cohesive soils					
Heavy sandy loam		3.3	3.9	4.6	4.9
Light		3.1	3.9	4.6	4.9
Loess soils in the conditions of finished settlement		2.6	3.3	3.9	4.3

Notes:

(1) Bed materials may move if velocities are higher than the nonscour velocities.

(2) Mean depth is calculated by dividing the cross-sectional area of the waterway by the top width of the water surface. If the waterway can be subdivided into a main channel and an overbank area, the mean depths of the channel and the overbank should be calculated separately. For example, if the size of moving material in the main channel is desired, the mean depth of the main channel is calculated by dividing the cross-sectional area of the main channel by the top width of the main channel.

Table 855.2C

Guide for Anticipated Service Life Added to Steel Pipe by Abrasive Resistant Protective Coating

Flow Velocity (ft/s)	Channel Materials	Bituminous Coating (yrs.) (hot-dipped)	Bituminous Coating & Paved Invert (yrs.)	Polymerized Asphalt (yrs.) (hot-dipped)	Polymeric Sheet Coating (yrs.)	Polyethylene (CSSRP) (yrs.)
< 5 (1)	Non-Abrasive	8	15	*	*	*
≥ 3 – ≤ 8 (2)	Abrasive	6-0	15-2	30-5	30-5	*
> 8 – ≤ 12	Abrasive	0	2-0	5-0	5-0	70-35
> 12 – ≤ 15	Abrasive	**	**	**	**	35-8***
> 15 – ≤ 20 or > 12 – ≤ 20	Abrasive & heavy bedloads	****	****	****	****	****

* Provides adequate abrasion resistance to meet or exceed a 50-year design service life.

** Abrasive resistant protective coatings not recommended, increase steel thickness to 10 gage.

*** Not recommended above 14 fps flow velocity.

**** Contact District Hydraulics Branch. See Table 855.2F.

Notes:

(1) Where there are increased velocities with no bedload (e.g. urban storm drain systems or culverts ≤ 30" dia.), higher velocities may be applicable.
(2) Where there are increased velocities with minor bedload (e.g. urban storm drain systems or culverts ≤ 30" dia.), higher velocities may be applicable.
(3) Range of additional service life commensurate with flow velocity range.

Table 855.2D

Guide for Anticipated Wear to Metal Pipe by Abrasive Channel Materials

Flow Velocity (ft/s)	Channel Materials	Anticipated Wear (mils/yr)		
		Plain Galvanized	Aluminized Steel (Type 2)	Aluminum**
	Non-Abrasive	0*	0*	0
≥ 3 – ≤ 8	Abrasive	0*	0*	0 – 1.5
> 8 – ≤ 12	Abrasive	0.5 – 1	0.5 – 1	1.5 – 3
> 12 – ≤ 15	Abrasive	1 – 3.5	1 – 3.5	3 – 10.5
> 15 – ≤ 20 or > 12 – ≤ 20	Abrasive & Heavy bedloads	2.5 – 10	2.5 – 10	7.5 – 30

* Refer to California Test 643 and Figure 855.3B.

** Refer to Figure 855.3A.

Note:

1 mil = 0.001"

Table 855.2E

Relative Abrasion Resistance Properties of Pipe and Lining Materials*

Material	Relative Wear (dimensionless)
Steel	1
Aluminum	1.5 – 3
PVC	2
Polyester Resin (CIPP)	2.5 – 4
HDPE	4 – 5
Concrete (RCP 4000 – 7000 psi)	75 – 100
Calcium Aluminate (Mortar)	6
Basalt Tile	1
Polyethylene (CSSRP)	1 – 2

* Evaluation of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA/CA/TL-CA01-0173 (2007).

Table 855.2F

Guide for Minimum Material Thickness of Abrasive Resistant Invert Protection to Achieve 50 Years of Maintenance-Free Service Life

Abrasion Level & Flow Velocity (ft/s)	Channel Materials	Concrete (in)	Steel Pipe & Plate (in)	Aluminum Pipe & Plate (in)	PVC (in)	HDPE (in)	CIPP (in)	Calcium Aluminate Abrasion Resistant Concrete (in)
Level 4 > 8 – ≤ 12	Abrasive	2 – 4	N/A	0.075 – 0.164	0.1	0.125 – 0.25	0.1 – 0.3	N/A
Level 5 > 12 – ≤ 15	Abrasive	4 – 13	0.052 – 0.18	**	0.1 – 0.35	0.25 – 0.875	0.3 – 0.70	N/A
Level 6 > 15 – ≤ 20 or > 12 – ≤ 20	Abrasive & Heavy bedloads	*	0.28 – 0.5 or 0.109 – 0.5	**	0.55 – 1.0*** or 0.25 – 1.0***	1.25 – 2.5 or 0.625 – 2.5	1 – 2 or 0.5 – 2	3 – 4

* For flow velocity > 12 ft/s ≤ 14 ft/s use 9" – 15". For > 14 ft/s use CRSP or other abrasion resistant layer special design with, or in lieu of concrete.

** Not recommended without invert protection.

*** Limited to 3" maximum stone size, otherwise PVC not recommended.

characteristics, i.e., volume, duration and frequency of stream flow in the culvert, will probably also have different abrasion levels. Table 855.2B can be used as a guide with Table 855.2A to determine the maximum size of material that can be moved through a pipe. Field observations of channel bed material both upstream and downstream from the pipe are extremely important for estimating the size range of transportable material in the channel.

855.3 Corrosion

Corrosion is the destructive attack on a pipe by a chemical reaction with the materials surrounding the pipe. Corrosion problems can occur when metal pipes are used in locations where the surrounding materials have excess acidity or alkalinity. The relative acidity of a substance is often expressed by its pH value. The pH scale ranges from 1 to 14, with 1 representing extreme acidity, and 14 representing extreme alkalinity, and 7 representing a neutral substance. The closer the pH value is to 7, the less potential the substance has for causing corrosion.

Corrosion is an electrolytic process and requires an electrolyte (generally moisture) and oxygen to proceed. As a result, it has the greatest potential for causing damage in soils that have a relative high ability to pass electric current. The ability of a soil to convey current is expressed as its resistivity in ohm-cm, and a soil with a low resistivity has a greater ability to conduct electricity. Very dry areas (e.g., desert environments) have a limited availability of electrolyte, and totally and continuously submerged pipes have limited oxygen availability. These extreme conditions (among others) are not well represented by AltPipe, and some adjustment in the estimated service life for pipes in these conditions should be made. See Index 857.2

Corrosion can also be caused by excessive acidity in the water conveyed by the pipe. Water pH can vary considerably between watersheds and seasons.

Because failure can occur at any point along the length of the pipe (e.g. tidal zones), the designer must look at the conditions and how they may vary along the pipe length - and select for input into AltPipe those conditions that represent the most severe situation along the length.

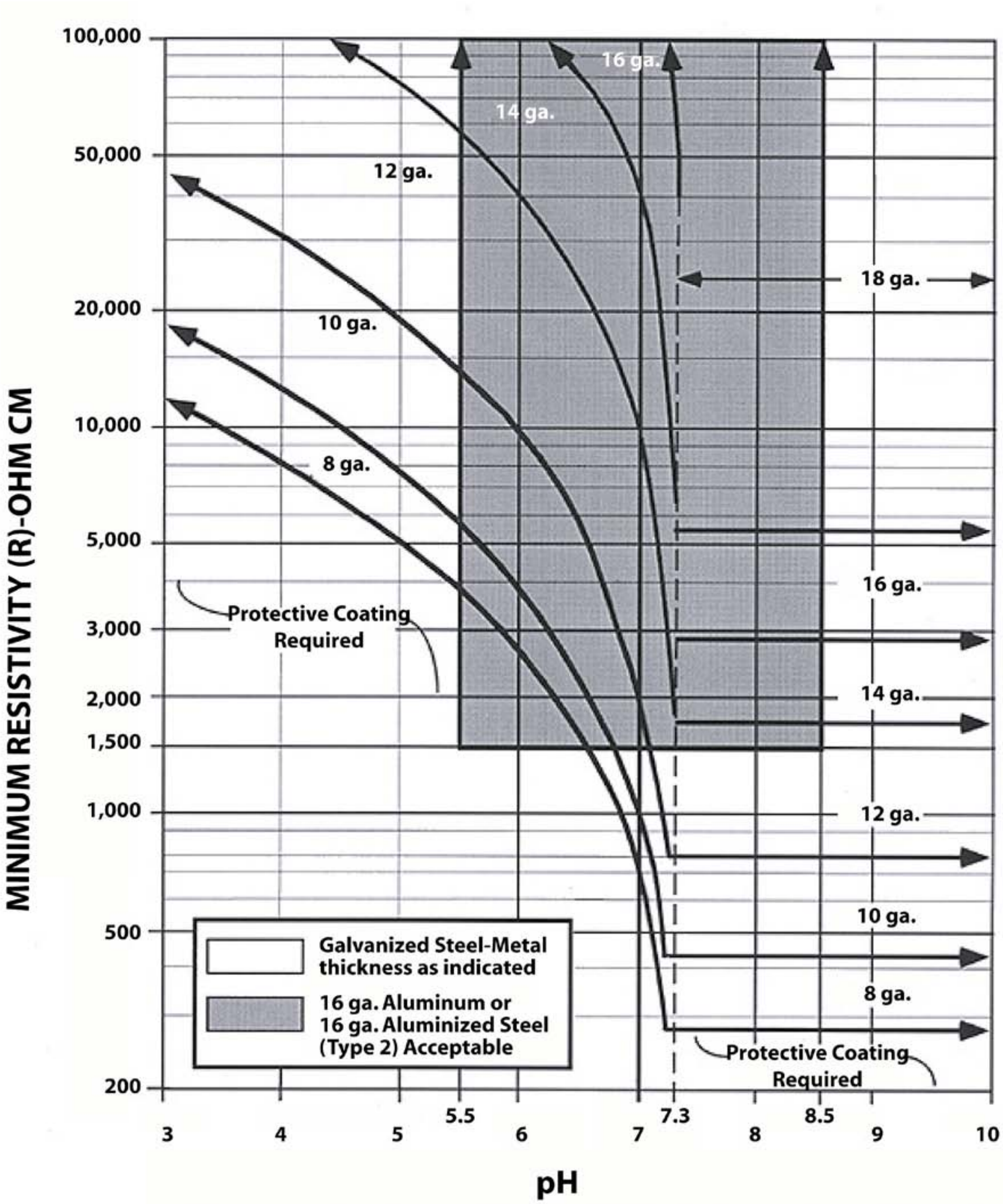
AltPipe operates based on some fairly basic assumptions for corrosion and minimum resistivity that are part of California Test 643. Altpipe will list all viable alternatives for achieving design service life. Where enhanced soilside corrosion protection is needed, aluminum or aluminized pipe (if within acceptable pH/min. resistivity ranges), bituminous coatings or polymeric sheet coating should be considered.

Aluminum, and the aluminum coating provided by Aluminized Steel (Type 2) pipe, corrodes differently than steel and will provide adequate durability to meet the 50-year service life criterion within the acceptable pH range of 5.5-8.5 and minimum resistivity greater than 1500 ohm-cm without need for specifying a thicker gauge or additional coating, whereas under the same range galvanized steel may need a protective coating or an increase in thickness to provide a 50-year maintenance-free service life (with respect to corrosion). Figure 855.3A should be used to determine the limitations on the use of corrugated aluminum pipe for various levels of pH and minimum resistivity. The minimum thickness (0.060 inch) of aluminum pipe obtained from the chart only satisfies corrosion requirements. Overfill requirements for minimum metal thickness must also be satisfied. The metal thickness of corrugated aluminum pipe should satisfy both requirements.

Figure 855.3A should be used to determine the minimum thickness and limitation on the use of corrugated steel and spiral rib pipe for various levels of pH and minimum resistivity. For example, given a soil environment with pH and minimum resistivity levels of 6.5 and 15,000 ohm-cm, respectively, the minimum thicknesses for the various metal pipes are: 1) 0.109 inch (12 gage) galvanized steel, 2) 0.064 inch (16 gage) aluminized steel (type 2) and 3) 0.060 inch (16 gage) aluminum. The minimum thickness of metal pipe obtained from the figure only satisfies corrosion requirements. Overfill requirements for minimum metal thickness must also be satisfied. The metal thickness of corrugated pipe and steel spiral rib pipe that satisfies both requirements should be used.

Figure 855.3B, "Chart for Estimating Years to Perforation of Steel Culverts," is part of a Standard California Department of Transportation Test Method derived from highway culvert investigations. This chart alone is not used for

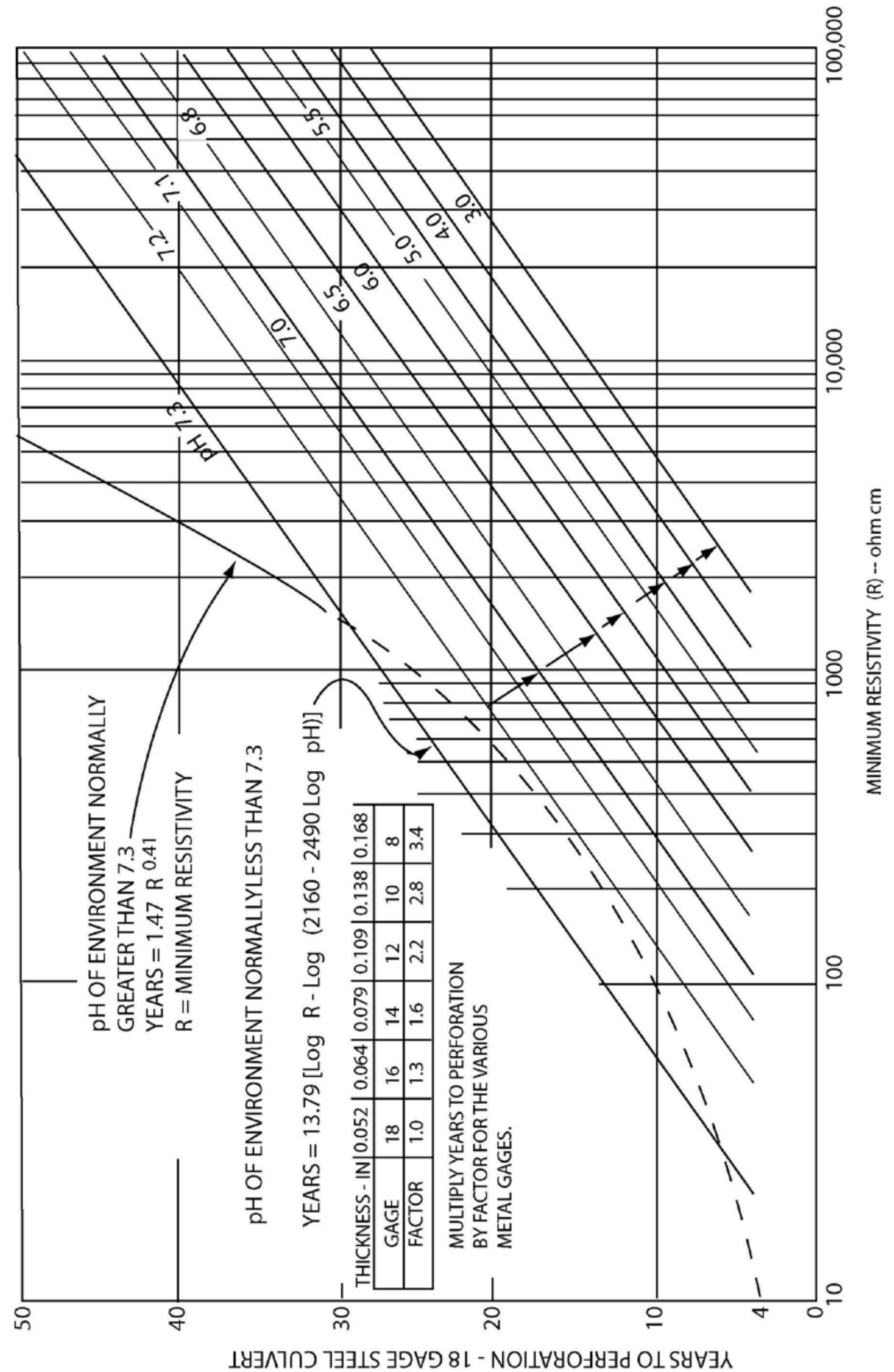
Figure 855.3A
Minimum Thickness of Metal Pipe
for 50 Year Maintenance-Free Service Life (2)



Notes:
1. For pH and minimum resistivity levels not shown refer to Fig. 854.3C steel pipes. (California Test 643)
2. Service life estimate are for various corrosive conditions only.
3. Refer to index 854.3(2) and 854.4(2) for appropriate selection of metal thickness and protection coating to achieve service life requirements.

Figure 855.3B

Chart for Estimating Years to Perforation of Steel Culverts



determining service life because it does not consider the effects of abrasion or overfill; it is for estimating the years to the first corrosion perforation of the wall or invert of the CSP.

855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates

Table 855.4A indicates the limitation on the use of concrete by acidity of soil and water. Table 855.4A is also a guide for designating cementitious material restrictions and water content restrictions for various ranges of sulfate concentrations in soil and water for all cast in place and precast construction of drainage structures.

For pH ranging between 7.0 and 3.0 and for sulfate concentrations between 1500 and 15,000 ppm, concrete mix designs conforming to the recommendations given in Table 855.4A should be followed. Higher sulfate concentrations or lower pH values may preclude the use of concrete or would require the designer to develop and specify the application of a complete physical barrier. Reinforcing steel can be expected to respond to corrosive environments similarly to the steel in CSP.

Table 855.4B provides a guide for minimum concrete cover requirements for various ranges of chloride concentrations in soil and water for all precast and cast in place construction of drainage structures.

(1) *RCP*. In relatively severe acidic, chloride or sulfate environments (either in the soil or water) as identified in the project Materials Report, the means for offsetting the effects of the corrosive elements is to either increase the cover over the reinforcing steel, increase the cementitious material content, or reduce the water/cementitious material ratio. The identified constituent concentration levels should be entered into AltPipe to verify what combinations of increased cover (in 1/4-inch intervals from 1 inch to a maximum of 1-1/2 inches), increased cementitious material content (in increments of 47 pounds from 470 pounds to a maximum of 564 pounds), will provide the necessary service life (typically 50 years). Per an agreement with Industry, the water to cementitious material ratio is set at 0.40. AltPipe is specifically programmed to provide RCP mix and cover designs that are compatible with industry practice, and are based on their agreements with Caltrans. For corrosive condition installations such as low pH (<4.5), Chlorides (>2,000 ppm) or Sulfates (> 2,000 ppm), the following service life (SL) equation provides the basis for RCP design in AltPipe:

$$SL = 10^3 \times 1.107^{Cc} \times Cc^{0.717} \times Dc^{1.22} \times (K+1)^{-0.37} \times W^{-0.631} - 4.22 \times 10^{10} \times pH^{-14.1} - 2.94 \times 10^{-3} \times S + 4.41$$

Where: S = Environmental sulfate content in ppm.

Cc = Sacks of cement (94 lbs each) per cubic yard of concrete.

Dc = Concrete cover in inches.

K = Environmental chloride concentration in ppm.

W = Water by volume as percentage of total mix.

pH = The measure of relative acidity or alkalinity of the soil or water. See Index 855.3.

Where the measured concentration of chlorides exceeds 2000 ppm for RCP that is placed in brackish or marine environments and where the high tide line is below the crown of the invert, the AltPipe input for chloride concentration will default to 25,000 ppm.

Contact the District Materials unit or the Corrosion Technology Branch in DES for design recommendations when in extremely corrosive conditions. Non-Reinforced concrete pipe is not affected by chlorides or stray currents and may be used in lieu of RCP with additional concrete cover and/or protective coatings for sizes 36" in diameter and smaller. See Index 852.1(4) and Table 855.4A. Where conditions occur that RCP designs as produced by AltPipe will not work, the Office of State Highway Drainage Design within the Division of Design should be contacted.

Table 855.4A

Guide for the Protection of Cast-In-Place and Precast Reinforced and Unreinforced Concrete Structures(5) Against Acid and Sulfate Exposure Conditions(1),(2)

Soil or Water pH	Sulfate Concentration of Soil or Water (ppm)	Cementitious Material Requirements (3)	Water Content Restrictions
7.1 to 14	0 to 1,500	Standard Specifications Section 90	No Restrictions
5.6 to 7.0	Greater Than 1,500 to 2,000	Standard Specifications Section 90	Maximum water-to-cementitious material ratio of 0.45
3 to 5.5(4)	Greater Than 2,000 to 15,000(4)	675 lb/cy minimum: Type II or Type V portland cement and required supplementary cementitious materials per Standard Specification 90-1.02H	Maximum water-to-cementitious material ratio of 0.40

Notes:

(1) Recommendations shown in the table for the cementitious material requirements and water content restrictions should be used if the pH and/or the sulfate conditions in Column 1 and/or Column 2 exists. Sulfate testing is not required if the minimum resistivity is greater than 1,000 ohm-cm.

(2) The table lists soil/water pH and sulfate concentration in increasing level of severity starting from the top of the table. If the soil/water pH and the sulfate concentration are at different levels of severity, the recommendation for the more severe level will apply. For example, a soil with a pH of 4.0, but with a sulfate concentration of only 1,600 ppm would require a minimum of 675 lb/cy of cementitious material. The maximum water-to-cementitious material ratio would be 0.40.

(3) Cementitious material shall conform to the provisions in Section 90of the Standard Specifications.

(4) Additional mitigation measures will be needed for conditions where the pH is less than 3 and/or the sulfate concentration exceeds 15,000 ppm. Mitigation measures may include additional concrete cover and/or protective coatings. For additional assistance, contact the Corrosion Technology Branch of Materials Engineering and Testing Services (METS) at 5900 Folsom Boulevard Sacramento, CA. 95819.

(5) Does not include RCP.

Table 855.4B

Guide for Minimum Cover Requirements for Cast-In-Place and Precast Reinforced Concrete Structures(3) for 50-Year Design Life in Chloride Environments

Chloride Concentration (ppm)			
500 to 2000	2001 to 5000	5001 to 10000	10000 +
1.5 in.(1)	2.5 in.(1)	3 in.(1)	4 in.(1)
1.5 in.(2)	1.5 in.(2)	2 in.(2)	3 in.(2)

Notes:

(1) Supplementary cementitious materials are required. Typical minimum requirement consists of 675#/cy minimum cementitious material with 75% by weight of Type II or Type V portland cement and 25% by weight of either fly ash or natural pozzolan. A maximum w/cm ratio of 0.40 is specified. Fly ash or natural pozzolan may have a CaO content of up to 10%. Specification S8-C04 provides requirements.

(2) Additional supplementary cementitious materials per the requirements of Specification S8-C04 are required in order to achieve the listed reduction in concrete cover.

(3) Does not include RCP.

855.5 Material Susceptibility to Fire

Fire can occur almost anywhere on the highway system. Common causes include forest, brush or grass fires that either enter the right-of-way or begin within it. Less common causes include spills of flammable liquids that ignite or vandalism. Storm drains, which are completely buried would typically be impacted by spills or vandalism. Because these are such low probability events, prohibitions on material placement for storm drains are not typically warranted.

Cross culverts and exposed overside drains are the placement types most subject to burning or melting and designers should consider either limiting the alternative pipe listing to non-flammable pipe materials or providing a non-flammable end treatment to provide some level of protection.

. Plastic pipe and pipes with coatings (typically of bituminous or plastic materials) are the most susceptible to damage from fire. Of the plastic pipe types which are allowed, PVC will self extinguish if the source of the fire is eliminated (i.e., if the grass or brush is consumed or removed) while HDPE can continue to burn as long as an adequate oxygen supply is present. Based on testing performed by Florida DOT, this rate of burning is fairly slow, and often self extinguished if the airflow was inhibited (i.e., pipe not aligned with prevailing wind or ends sheltered from air flow).

Due to the potential for fire damage, plastic pipe is not recommended for overside drain locations where there is high fire potential (large amounts of brush or grass or areas with a history of fire) and where the overside drain is placed or anchored on top of the slope.

Where similar high fire potential conditions exist for cross culverts, the designer may consider limiting the allowable pipe materials indicated on the alternative pipe listing to non-flammable material types, use concrete endwalls that eliminate exposure of the pipe ends, or require that the end of flammable pipe types be replaced with a length of non-flammable pipe material.

Topic 856 - Height of Fill

An essential aspect of pipe selection is the height of fill/cover over the pipe. This cover dissipates live loads from traffic, both during construction and after the facility is open to the public.

856.1 Construction Loads

See Standard Plan D88 for table of minimum cover for construction loads.

856.2 Concrete Pipe, Box and Arch Culverts

(1) *Reinforced Concrete Pipe.* See Standard Plan A62D and A62DA for the maximum height of overfill for reinforced concrete pipe, up to and including 120-inch diameter (or reinforced oval pipe and reinforced concrete pipe arch with equivalent cross-sectional area), using the backfill method or type shown. For oval shaped reinforced concrete pipe fill heights, see Standard Plan A62D and Indirect Design D-Load (Marsten/Spangler Method). Allowable cover for oval shaped reinforced concrete pipe is determined by using Method 2 (Note 8). See Standard Plan D79 and D79A for pre-cast reinforced concrete pipe Direct Design Method (pertains to circular pipe only).

The designer should be aware of the premises on which the tables on Standard Plan A62D, A62DA, D79 and D79A are computed as well as their limitations. The cover presupposes:

- That the bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover and pipe size required by the plans, and take into account the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert equal in magnitude to that of the adjoining material outside the trench.
- Subexcavation and backfill as required by the Standard Specifications where unyielding foundation material is encountered.

If the height of overfill exceeds the tabular values on Standard Plan A62D and A62DA a special design is required; see Index 829.2.

(2) *Concrete Box and Arch Culverts.* Single and multiple span reinforced concrete box culverts are completely detailed in the Standard Plans. For cast-in-place construction, strength classifications are shown for 10 feet and 20 feet overfills. See Standard Plan numbers D80, D81 and D82. Pre-cast reinforced concrete box culverts require a minimum of 1 foot overfill and limit fill height to 12 feet maximum. See Standard Plans D83A, D83B and A62G. For fill height design criteria for CIP Bottomless 3-sided rigid frame culverts see XS-Sheets 17-050-1, 2, 3, 4 and 5. Cast-in-place reinforced concrete arch culverts are no longer economically feasible structures and last appeared in the 1997 Standard Plans. Questions regarding fill height for concrete arch culverts or extensions should be directed to the Underground Structures Branch of DES - Structures Design.

856.3 Metal Pipe and Structural Plate Pipe

Basic Premise - To properly use the fill height design tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications and Standard Plan A62F, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

Limitations - In using the tables, the following restrictions must be kept in mind:

- The values given for each size of pipe constitute the maximum height of overfill or cover over the pipe for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. Where abrasive conditions are anticipated, additional metal thickness or invert treatments as stated under Index 852.4(5) and Index 852.6(2)(c) should be provided when required to fulfill the design service life requirements of Topic 855.
- Where needed, adequate provisions for corrosion resistance must be made to achieve the required design service life called for in the references mentioned herein.
- Table 856.3D shows the limit of heights of cover for corrugated steel pipe arches based on the supporting soil sustaining a factored bearing pressure varying between 3.38 tons per square feet to 3.55 tons per square feet. Table 856.3J shows similar values for corrugated aluminum pipe arches.
- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- Tables 856.3N & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a factored bearing pressure of 6 tons per square foot at the corners.

Special Designs.

- If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for pipe arches are based, a special design prepared by DES - Structures Design is required. See index 829.2.
- Non-standard pipe diameters and arch sizes are available. Loading capacity of special designs needs to be verified with the Underground Structures Branch of DES - Structures Design.
- Aluminum pipe fill height tables are based on use of H-32 temper aluminum. If use of aluminum is necessary and greater structural capacity is required, H-34 temper can be specified. Contact Underground Structures branch of DES-Structures Design for calculation of allowable fill height.

(1) *Corrugated Steel Pipe and Pipe Arches, Steel Spiral Rib Pipe, Structural Steel Plate Pipe and Structural Steel Plate Pipe Arches.* The allowable overfill heights for corrugated steel pipe and pipe arches for the various diameters

Table 856.3A

Corrugated Steel Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)					
	Metal Thickness (in)					
	0.052 (18 ga.)	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
	2⅔" x ½" Corrugations					
12-15	118	148	--	--	--	--
18	99	124	--	--	--	--
21	85	106	132	--	--	--
24	74	93	116	--	--	--
30	59	74	93	130	--	--
36	49	62	77	108	139	--
42	42	53	66	93	119	--
48	--	46	58	81	104	128
54	--	--	51	72	93	113
60	--	--	--	65	83	102
66	--	--	--	--	76	93
72	--	--	--	--	70	85
78	--	--	--	--	--	75
84	--	--	--	--	--	65
	3" x 1" Corrugations					
48	--	53	67	93	120	147
54	--	47	59	83	107	131
60	--	42	53	75	96	118
66	--	39	48	68	87	107
72	--	35	44	62	80	98
78	--	33	41	57	74	91
84	--	30	38	53	69	84
90	--	28	35	50	64	78
96	--	--	33	47	60	74
102	--	--	31	44	56	69
108	--	--	--	41	53	65
114	--	--	--	39	50	62
120	--	--	--	37	48	59

NOTE:
(1) When flow velocity exceeds 5 ft/s under abrasive conditions, thicker metal may be required.

Table 856.3B

Corrugated Steel Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft) Metal Thickness (in)			
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)
	5" x 1" Corrugations			
48	47	59	83	--
54	42	53	74	95
60	38	47	66	86
66	34	43	60	78
72	31	39	55	71
78	29	36	51	66
84	27	34	47	61
90	25	31	44	57
96	--	29	41	53
102	--	28	39	50
108	--	--	37	47
114	--	--	35	45
120	--	--	33	43

NOTE:
(1) When flow velocity exceeds 5 ft/s under abrasive conditions, thicker metal may be required.

Table 856.3C

Corrugated Steel Pipe
2⅔" x ½" Annular Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)				
	Metal Thickness (in)				
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
18	54	--	--	--	--
21	46	--	--	--	--
24	40	44	--	--	--
30	32	35	--	--	--
36	27	29	38	--	--
42	30	41	65	68	--
48	26	36	57	59	62
54	--	32	50	53	55
60	--	--	45	47	50
66	--	--	--	43	45
72	--	--	--	39	41
78	--	--	--	--	38
84	--	--	--	--	35

NOTE:
(1) When flow velocity exceeds 5 ft/s under abrasive conditions, thicker metal may be required.

Table 856.3D

Corrugated Steel Pipe Arches
2⅔" x ½" Helical or Annular Corrugations

Span-Rise (in)	Factored Bearing Demand (tons/ft^2)	Minimum Corner Radius (in)	MAXIMUM HEIGHT OF COVER (ft) Metal Thickness (in) 0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
21 x 15	3.50	4 1/8	10	--	--	--
24 x 18	3.38	4 7/8	10	--	--	--
28 x 20	3.49	5 1/2	10	--	--	--
35 x 24	3.49	6 7/8	10	--	--	--
42 x 29	3.49	8 1/4	10	--	--	--
49 x 33	3.49	9 5/8	10	--	--	--
57 x 38	3.55	11	--	10	--	--
64 x 43	3.54	12 3/8	--	10	--	--
71 x 47	3.54	13 3/4	--	--	10	--
77 x 52	3.49	15 1/8	--	--	--	10
83 x 57	3.45	16 1/2	--	--	--	10

NOTES:
(1) When flow velocity exceeds 5 ft/s under abrasive conditions, thicker metal may be required.
(2) Cover limited by corner soil bearing pressure as shown.

Table 856.3E

Steel Spiral Rib Pipe
¾" x 1" Ribs at 11½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)
24	44	62	105
30	36	50	84
36	30	42	70
42	25	36	60
48	22	31	53
54	20	28	47
60	--	25	42
66	--	22	38
72	--	21	35
78	--	--	32
84	--	--	30
90	--	--	28
96	--	--	--

NOTE:

(1) When flow velocity exceeds 5 ft/s under abrasive conditions, thicker metal may be required.

Table 856.3F

Steel Spiral Rib Pipe
¾" x 1" Ribs at 8½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)
24	59	83	137
30	48	66	110
36	40	55	92
42	34	47	78
48	30	41	69
54	26	37	61
60	24	33	55
66	21	30	50
72	20	27	46
78	--	25	42
84	--	23	39
90	--	--	36
96	--	--	34
102	--	--	32
108	--	--	30
114	--	--	--

NOTE:

(1) When flow velocity exceeds 5 ft/s under abrasive conditions, thicker metal may be required.

Table 856.3G

Steel Spiral Rib Pipe
¾” x ¾" Ribs at 7½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)			
	Metal Thickness (in)			
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)
24	61	85	141	205
30	49	68	113	164
36	40	57	94	137
42	35	48	81	117
48	30	42	71	103
54	27	38	63	91
60	--	34	57	82
66	--	31	51	75
72	--	--	47	68
78	--	--	43	63
84	--	--	40	59
90	--	--	--	55

NOTE:

(1) When flow velocity exceeds 5 ft/s under abrasive conditions, thicker metal may be required.

Table 856.3H
Corrugated Aluminum Pipe
Annular Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)				
	Metal Thickness (in)				
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
	2⅔" x ½" Corrugations				
12	43	43	--	--	--
15	35	34	60	--	--
18	29	29	50	--	--
21	25	25	43	--	--
24	21	21	37	39	--
30	--	17	30	31	--
36	--	14	25	26	--
42	--	--	43	45	--
48	--	--	38	40	41
54	--	--	34	35	36
60	--	--	--	32	33
66	--	--	--	--	30
72	--	--	--	--	27
	3" x 1" Corrugations				
30	32	40	54	81	--
36	26	33	45	68	88
42	23	28	39	58	75
48	20	25	34	51	66
54	17	22	30	45	59
60	16	20	27	41	53
66	14	18	24	37	48
72	13	16	22	34	44
78	--	15	21	31	40
84	--	--	19	29	38
90	--	--	18	27	35
96	--	--	17	25	33
102	--	--	--	24	31
108	--	--	--	22	29
114	--	--	--	--	28
120	--	--	--	--	26

NOTE:
(1) Not recommended under abrasive conditions.

Table 856.3I
Corrugated Aluminum Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)				
	Metal Thickness (in)				
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
	2⅔" x ½" Corrugations				
12	112	140	--	--	--
15	90	112	156	--	--
18	75	93	130	--	--
21	64	80	112	--	--
24	56	70	98	126	--
30	--	56	78	101	--
36	--	47	65	84	--
42	--	--	56	72	--
48	--	--	49	63	77
54	--	--	43	56	68
60	--	--	--	46	58
66	--	--	--	--	47
72	--	--	--	--	37
	3" x 1" Corrugations				
30	51	65	90	121	--
36	43	54	75	101	118
42	37	46	64	86	102
48	32	40	56	76	89
54	28	36	50	67	79
60	26	32	45	60	71
66	23	29	41	55	65
72	21	27	37	50	59
78	--	25	35	46	55
84	--	--	32	43	51
90	--	--	30	40	47
96	--	--	28	38	44
102	--	--	--	35	42
108	--	--	--	33	39
114	--	--	--	--	36
120	--	--	--	--	32

NOTE:
(1) Not recommended under abrasive conditions.

Table 856.3J

Corrugated Aluminum Pipe Arches
2⅔" x ½" Helical or Annular Corrugations

Span-Rise (in)	Factored Bearing Demand (tons/ft^2)	Minimum Corner Radius (in)	MAXIMUM HEIGHT OF COVER (ft)				
			Metal Thickness (in)				
			0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
17 x 13	3.34	3 1/2	10	--	--	--	--
21 x 15	3.49	4 1/8	10	--	--	--	--
24 x 18	3.38	4 7/8	10	--	--	--	--
28 x 20	3.49	5 1/2	--	10	--	--	--
35 x 24	3.49	6 7/8	--	10	--	--	--
42 x 29	3.49	8 1/4	--	--	10	--	--
49 x 33	3.49	9 5/8	--	--	10	--	--
57 x 38	3.55	11	--	--	--	10	--
64 x 43	3.54	12 3/8	--	--	--	10	--
71 x 47	3.54	13 3/4	--	--	--	--	10

NOTES:
(1) Cover is limited by corner soil bearing pressure as shown.
(2) Not recommended under abrasive conditions.

Table 856.3K

Aluminum Spiral Rib Pipe
¾" x 1” Ribs at 11½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)
24	22	31	50
30	18	24	40
36	15	20	33
42	--	17	29
48	--	--	25
54	--	--	22
60	--	--	20
66	--	--	--
72	--	--	--

NOTE:
(1) Not recommended under abrasive conditions.

Table 856.3L

Aluminum Spiral Rib Pipe
¾" x ¾" Ribs at 7½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft) Metal Thickness (in)		
	0.60 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)
24	30	41	66
30	24	33	53
36	20	27	44
42	--	23	38
48	--	--	33
54	--	--	29
60	--	--	26
66	--	--	--
72	--	--	--

NOTE:
(1) Not recommended under abrasive conditions.

Table 856.3M
Structural Steel Plate Pipe
6" x 2" Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)							
	Metal Thickness (in)							
	0.110 (12 ga.)	0.140 (10 ga.)	0.170 (8 ga.)	0.218 (5 ga.)	0.249 (3 ga.)	0.280 (1 ga.)	0.318 (0 ga.)	0.380 (000 ga.)
60	42	60	79	105	128	140	223	268
66	38	55	71	99	116	127	203	243
72	35	50	65	91	107	116	186	223
77	32	47	61	85	100	109	174	209
84	30	43	56	78	92	100	160	192
90	28	40	52	72	85	93	149	179
96	26	37	49	68	80	87	140	168
102	24	35	46	64	75	82	132	158
108	23	33	44	60	71	78	124	149
114	22	31	41	57	67	74	118	141
120	21	30	39	54	64	70	112	134
126	20	28	37	52	61	67	107	128
132	19	27	36	49	58	63	102	122
138	18	26	34	47	56	61	91	117
144	17	25	33	45	53	58	93	112
150	16	24	31	43	51	56	89	108
156	16	23	30	42	49	54	86	103
162	15	22	29	40	47	52	83	100
168	15	21	28	39	46	50	80	96
174	14	20	27	37	44	48	77	93
180	14	20	26	36	43	46	75	90
186	13	19	25	35	41	45	72	87
192	--	18	24	34	40	44	70	84
198	--	18	24	33	39	42	68	81
204	--	17	23	32	38	41	66	79
210	--	17	22	31	36	40	64	77
216	--	--	22	30	35	39	62	75
222	--	--	21	29	34	38	60	73
228	--	--	20	28	34	37	59	71
234	--	--	20	28	33	36	57	69
240	--	--	--	27	32	35	56	67
246	--	--	--	26	31	34	54	65
252	--	--	--	26	30	33	53	64

NOTE:
(1) When flow velocities exceeds 5 ft/s under abrasive conditions thicker metal may be required.

Table 856.3N

Structural Steel Plate Pipe Arches
6" x 2" Corrugations

MAXIMUM HEIGHT OF COVER (ft)			
		Factored Corner Soil Bearing – 6 tons/ft^2	
Span	Rise	Metal Thickness (in)	
		0.110 (12 ga.)	0.140 (10 ga.)
		18" Corner Radius	
6'-1"	4'-7"	21	--
7'-0"	5'-1"	18	--
7'-11"	5'-7"	16	--
8'-10"	6'-1"	14	--
9'-9"	6'-7"	13	--
10'-11"	7'-1"	12	--
		31" Corner Radius	
13'-3"	9'-4"	17	--
14'-2"	9'-10"	16	--
15'-4"	10'-4"	13	--
16'-3"	10'-10"	12	--
17'-2"	11'-4"	12	--
18'-1"	11'-10"	11	--
19'-3"	12'-4"	--	10
19'-11"	12'-10"	--	10
20'-7"	13'-2"	--	10

NOTES:
(1) For intermediate sizes, the depth of cover may be interpolated.
(2) The 31-inch corner radius arch should be specified when conditions will permit it use.

Table 856.3O

Structural Aluminum Plate Pipe
9" x 2½" Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)						
	Metal Thickness (in)						
	0.100	0.125	0.150	0.175	0.200	0.225	0.250
60	27	40	52	62	71	81	90
66	24	36	48	56	65	73	82
72	22	33	44	51	59	67	75
77	21	31	41	48	55	63	70
84	19	28	37	44	51	58	64
90	18	26	35	41	47	54	60
96	17	25	33	38	44	50	56
102	16	23	31	36	42	47	53
108	15	22	29	34	39	45	50
114	14	21	27	32	37	42	47
120	13	20	26	31	35	40	45
126	13	19	25	29	34	38	43
132	12	18	24	28	32	36	41
138	11	17	23	27	31	35	39
144	--	16	22	25	29	33	37
150	--	16	21	24	28	32	36
156	--	15	20	23	27	31	35
162	--	--	19	23	26	30	33
168	--	--	18	22	25	29	32
174	--	--	18	21	24	28	31
180	--	--	--	20	23	27	30
186	--	--	--	20	23	26	29
192	--	--	--	--	22	25	28
198	--	--	--	--	21	24	27
204	--	--	--	--	--	23	26
210	--	--	--	--	--	23	26
216	--	--	--	--	--	22	25
222	--	--	--	--	--	--	24
228	--	--	--	--	--	--	23

NOTE:
(1) Not recommended under abrasive conditions.

Table 856.3P

Structural Aluminum Plate Pipe Arches
9" x 2½" Corrugations

		MAXIMUM HEIGHT OF COVER (ft)					
		Factored Corner Soil Bearing – 6 tons/ft^2					
		Metal Thickness (in)					
Span	Rise	0.100	0.125	0.150	0.175	0.200	0.225
6'-7"	5'-8"	20	--	--	--	--	--
7'-9"	6'-0"	17	--	--	--	--	--
8'-10"	6'-4"	15	--	--	--	--	--
9'-11"	6'-8"	13	--	--	--	--	--
10'-3"	6'-9"	13	19	--	--	--	--
11'-1"	7'-0"	12	18	20	--	--	--
12'-3"	7'-3"	11	16	18	--	--	--
12'-11"	7'-6"	10	15	17	--	--	--
13'-1"	8'-2"	10	15	17	--	--	--
13'-11"	8'-5"	9	14	16	--	--	--
14'-0"	8'-7"	9	14	16	--	--	--
14'-8"	9'-8"	--	13	15	--	--	--
15'-7"	10'-2"	--	12	13	--	--	--
16'-1"	10'-4"	--	12	13	--	--	--
16'-9"	10'-8"	--	--	12	--	--	--
17'-9"	11'-2"	--	--	--	11	--	--
18'-8"	11'-8"	--	--	--	11	--	--
19'-10"	12'-1"	--	--	--	--	10	--
20'-10"	12'-7"	--	--	--	--	--	9
21'-6"	12'-11"	--	--	--	--	--	9

NOTES:
(1) Not recommended under abrasive conditions.
(2) 31 inch Corner Radius

or arch sizes and metal thickness are shown on Tables 856.3A, B, C & D. For steel spiral rib pipe, overfill heights are shown on Tables 856.3E, F, G & H. Table 856.3G gives the allowable overfill height for composite steel spiral rib pipe.

For structural steel plate pipe and structural steel plate pipe arches, overfill heights are shown on Tables 856.3M & N. For maximum height of fill over structural steel plate vehicular undercrossings, see Standard Plan B14-1.

(2) *Corrugated Aluminum Pipe and Pipe Arches, Aluminum Spiral Rib Pipe and Structural Aluminum Plate Pipe and Structural Aluminum Plate Pipe Arches.* The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thickness are shown on Tables 856.3H, I & J. For aluminum spiral rib pipe, overfill heights are shown on Tables 856.3K & L.

For structural aluminum plate pipe and structural aluminum plate pipe arches, overfill heights are shown on Tables 856.3O, & P.

856.4 Plastic Pipe

The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5. To properly use the plastic pipe height of fill table, the designer should be aware of the basic premises on which the table is based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications and Standard Plan A62F, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That corrugated high density polyethylene (HDPE) pipe that is greater than 48" in size shall be backfilled with cementitious (slurry cement, CLSM or concrete) backfill.
- That where cementitious or flowable backfill is used for structural backfill, the backfill shall be placed to a level not less than 12 inches above the crown of the pipe.

Table 856.4

Thermoplastic Pipe Fill Height Tables

High Density Polyethylene (HDPE) Corrugated Pipe - Type S

Size (in)	Maximum Height of Cover (ft)
12	15
15	15
18	15
24	15
30	15
36	15
42	15
48	15
54	15
60	15

High Density Polyethylene (HDPE) Corrugated Pipe - Type C

Size (in)	Maximum Height of Cover (ft)
12	5
15	5
18	5
24	5

Polyvinyl Chloride (PVC) Dual Wall Corrugated Pipe

Size (in)	Maximum Height of Cover (ft)
12	35
15	35
18	35
21	35
24	35
30	35
36	35

- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.
- That the average water table elevation is at or below the pipe springline.
- Corrugated HDPE pipe, Type C is recommended for placement only outside the roadbed where vehicular loading is unlikely (e.g., overside drains, medians) unless cementitious backfill is specified.

856.5 Minimum Height of Cover

Table 856.5 gives the minimum thickness of cover required for design purposes over pipes and pipe arches. For construction purposes, a minimum cover of 6 inches greater than the roadway structural section is desirable for all types of pipe.

Where cover heights above culverts are less than the values shown in Table 856.5, stress reducing slab details available from the Headquarters Design drainage detail library using the following web address may be used: http://onramp.dot.ca.gov/hq/design/drainage/library.php

Where cover heights are less than the values shown in the stress reducing slab details, contact Office of State Highway Drainage Design or the Underground Structures Branch of DES - Structures Design.

Topic 857 - Alternate Materials

857.1 Basic Policy

When two or more materials meet the design service life, and structural and hydraulic requirements, the plans and specifications must provide for alternative pipes, pipe arches, overside drains, and underdrains to allow for optional selection by the contractor. See Index 114.3 (2).

(1) *Allowable Alternatives*. A table of allowable alternative materials for culverts, drainage systems, overside drains, and subsurface drains is included as Table 857.2. This table also identifies the various joint types described in Index 854.1(1) that should be used for the different types of installations.

(2) *Design Service Life*. Each pipe type selected as an alternative must have the appropriate protection as outlined in Topic 852 to assure that it will meet the design service life requirements specified in Topic 855. The maximum height of cover must be in accordance with the tables included in Topic 856.

(3) *Selection of a Specific Material Type*. In the cases listed below, the selection of a specific culvert material must be supported by a complete analysis based on the foregoing factors. All pertinent documentation should be placed on file in the District.

- Where satisfactory performance for a life expectancy of 25 or 50 years, as defined under design service life, cannot be obtained with certain materials by reason of highly corrosive conditions, severe abrasive conditions, or critical structural and construction requirements.
- For individual drainage systems such as roadway drainage systems or culverts which operate under hydrostatic pressure or culverts governed by hydraulic considerations and which would require separate design for each culvert type.
- When alterations or extensions of existing systems are required, the culvert type may be selected to match the type used in the existing system.

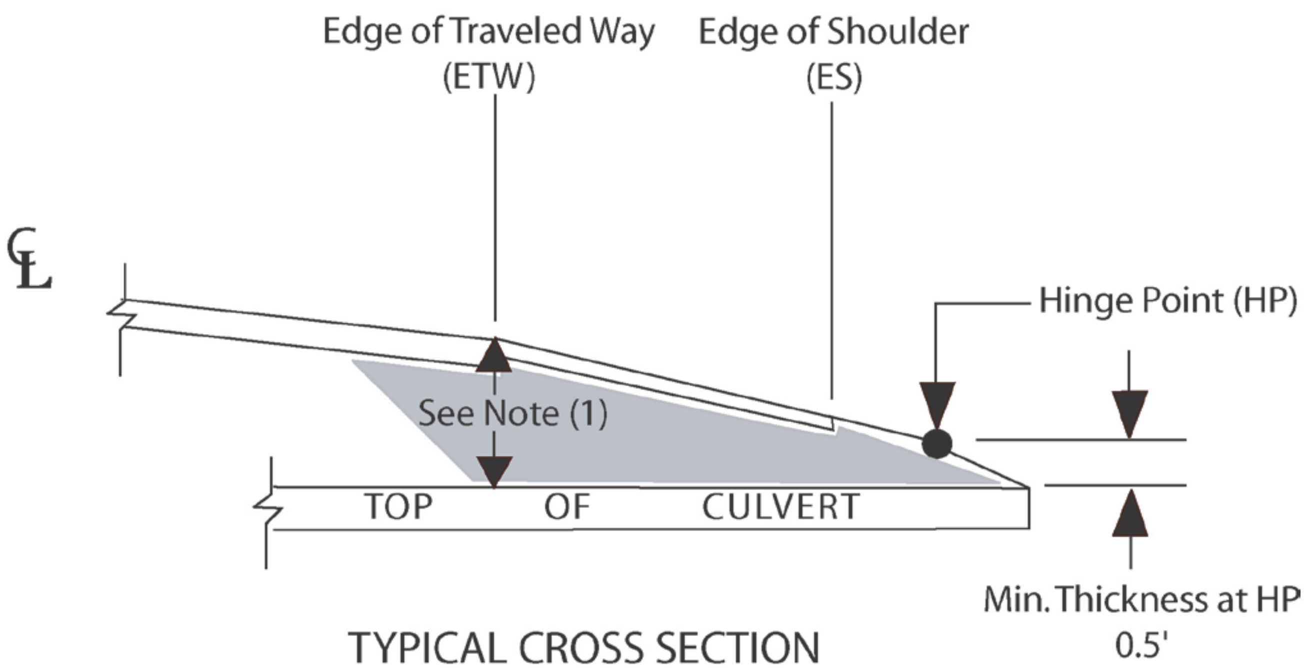
857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe

These instructions are general guidelines for alternative pipe culvert selection using the AltPipe computer program that is located on the Headquarters Division of Design alternative pipe culvert selection website at the following web address:
http://www.dot.ca.gov/hq/oppd/altpipe.htm.

AltPipe is a web-based tool that may be used to assist materials engineers and designers in the appropriate selection of pipe materials for culvert and storm drain applications. The computations performed by AltPipe are based on the procedures and California Test Methods described in this

Table 856.5

Minimum Thickness of Cover for Culverts



MINIMUM THICKNESS OF COVER AT ETW							
Corrugated Metal Pipes and Pipe Arches	Steel Spiral Rib Pipe	Aluminum Spiral Rib Pipe, S ≤ 48"	Aluminum Spiral Rib Pipe, S > 48"	Structural Plate Pipe	Reinforced Concrete Pipe (RCP) Under Rigid Pavement	RCP Under Flexible Pavement or Unpaved	Plastic Pipes
S/8 or 24" Min.	S/4 or 24" Min.	24" Min.	S/2.75 or 24" Min.	S/8 or 24" Min.	12" Min.	(Max Outside Dimension)/8 or 24" Min.	S/8 or 24" Min.

Notes:
(1) Minimum thickness of cover is measured at ultimate or failure edge of traveled way.
(2) Table is for HL-93 live load conditions only.
(3) "S" in the table is the maximum inside diameter or span of a section.

Chapter. AltPipe is not a substitute for the appropriate use of engineering judgment as conditions and experience would warrant. AltPipe establishes uniform procedures to assist the designer in carrying out the majority of the alternative pipe culvert selection functions of the Department, and is neither intended as, nor does it establish, a legal standard for these functions. Implementation of the results and output of this program is solely at the discretion of the user. The user is encouraged to first read the two informational links on the website titled 'Get More Information' and 'How to use Altpipe' prior to using the program.

Each alternative material selected for a drainage facility must provide the required design service life based on physical and structural factors, be of adequate size to satisfy the hydraulic design, and require the minimum of maintenance and construction cost for each site condition.

Step 1. Obtain the results of soil and water pH, resistivity, sulfate and chloride tests, proposed design life of culverts and make determination if any of the outfalls are in salty or brackish water. The Materials Report should include proposed design life and recommendations for pipe material alternatives. See Indexes 114.2 (3) and 114.3 (2).

Step 2. Obtain hydraulic studies and location data for pipe minimum sizes, and expected Q2-5 flow velocities. For pipes operating under outlet control, a critical element of pipe selection is the Manning's internal roughness value used in the hydraulic design. It is important to independently verify the roughness used in the design is applicable for the selected alternate materials from AltPipe. Rougher pipes may require larger sizes to provide adequate hydraulic capacities and need steeper slopes to produce desired cleaning velocities, usually however, pipe slope is maintained, and the only variable provided on the plans is pipe size.

Step 3. Determine the abrasion level from Table 852.2A from the maximum size of material that can be moved through a pipe, the expected Q2-5 flow velocities, and Table 855.2B. Field observations of channel bed material both upstream and downstream are recommended.

Step 4. Determine the maximum fill height.

Step 5. Using the AltPipe computer program that is located on the Headquarters Division of Design alternative pipe culvert selection website enter:

- Pipe diameter
- Maximum fill height
- Design service life
- pH
- Minimum resistivity
- Sulfate concentration
- Chloride concentration (for values greater than 2000, check boxes if end of culvert is exposed to brackish conditions and high tide line is below the crown of the culvert)
- Abrasion level
- 2-5-year Storm Flow Velocity (ft/sec)

Repeat step 5 as necessary and save each pipe in worksheet as needed and go to the final summary upon completion.

Step 6. The following alternatives are not included in AltPipe and will not be provided in the output Alternative pipe list: all non-circular shapes (arches, boxes, etc.), non reinforced concrete pipe (NRCP) and non-standard new products. Check Materials and Hydraulics reports and verify if any of these alternatives were recommended and supplement the AltPipe final summary accordingly. For reinforced concrete pipe (RCP), box (RCB) and arch (RCA) culverts, maintenance-free service life, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of exposed reinforcement at any point on the culvert. Changes in the design may be required in relatively severe acidic, chloride or sulfate environments. The levels of these constituents (either in the soil or water) will need to be identified in the project Materials or Geotechnical Design Report. The adopted procedure consists of a formula that the constituent concentrations are entered into in order to determine a pipe service life. The means for offsetting the affects of the corrosive elements is to increase the cover over the reinforcing steel, increase the cement content, or reduce the water/cement ratio.

Table 857.2

Allowable Alternative Materials

Type of Installation	Service Life (yrs)[1]	Allowable Alternatives	Joint Type Standard	Joint Type Positive	Joint Type Downdrain
Culverts & Drainage Systems	50	ASSRP, ASRP, CAP, CASP, CSSRP, CIPCP, CSP, NRCP, SAPP, SSPP, SSRP, RCP, RCB, PPC	X	X	--
Overside Drains	50	CAP, CASP, CSP, PPC	--	--	X
Underdrains	50	PAP, PSP, PPET, PPVCP	X	--	--
Arches (Culverts & Drainage Systems)	50	ACSPA, CAPA, CSPA, RCA, SAPPA, SSPPA, SSPA	X	X	--

<u>LEGEND</u>

ACSPA - Aluminized Corrugated Steel Pipe Arch
ASSRP - Aluminized Steel Spiral Rib Pipe
ASRP - Aluminum Spiral Rib Pipe
CAP - Corrugated Aluminum Pipe
CAPA - Corrugated Aluminum Pipe Arch
CSSRP - Composite Steel Spiral Rib Pipe
CASP - Corrugated Aluminized Steel Pipe, Type 2
CIPCP - Cast-in-Place Concrete Pipe
CSP - Corrugated Steel Pipe
CSPA - Corrugated Steel Pipe Arch
NRCP - Non-Reinforced Concrete Pipe
PAP - Perforated Aluminum Pipe
PPC - Plastic Pipe Culvert
PPET - Perforated Polyethylene Tubing
PPVCP - Perforated Polyvinyl Chloride Pipe
PSP - Perforated Steel Pipe
RCA - Reinforced Concrete Arch
RCB - Reinforced Concrete Box
RCP - Reinforced Concrete Pipe
SAPP - Structural Aluminum Plate Pipe
SAPPA - Structural Aluminum Plate Pipe Arch
SSPA - Structural Steel Plate Arch
SSPP - Structural Steel Plate Pipe
SSPPA - Structural Steel Plate Pipe Arch
SSRP - Steel Spiral Rib Pipe
X - Permissible Joint Type for the Type of installation Indicated

NOTE:
1. The design service life indicated for the various types of installations listed in the table may be reduced to 25 years in certain situations. Refer to Index 855.1 for a discussion of service life requirements.

Step 7. Table 855.2C constitutes a guide for abrasive resistant coatings in low to moderate abrasive conditions for metal pipe (i.e., Levels 1 through 5 in Table 855.2A) and is included in AltPipe. Table 855.2F constitutes a guide for minimum material thickness of abrasive resistant invert protection to achieve 50 years of maintenance-free service life in moderate to highly abrasive conditions (i.e., Levels 4 through 6 in Table 855.2A) and was not programmed into AltPipe. If pipe material thickness does not meet service life due to abrasive conditions, consideration for invert protection should be made using Table 855.2F as a guide.

857.3 Alternative Pipe Culvert (APC) and Pipe Arch Culvert List

Because of the difference in roughness coefficients between various materials, it may be necessary to specify a different size for each allowable material at any one location. In this event, it is recommended that the material with the smallest dimension be listed as the alternative size. Refer to Drafting and Plans Manual for standard format to be used.

There may be situations where there is a different set of alternatives for the same nominal size of alternative drainage facilities. In this case the different sets of the same nominal size should be further identified by different types, for example, 18-inch alternative pipe culvert (Type A), 18-inch alternative pipe culvert (Type B), etc. No attempt to correlate type designation between projects is necessary. The first alternative combination for each culvert size on each project should be designated as Type A, second as Type B, etc.

Since the available nominal sizes for pipe arches vary slightly between pipe arch materials, it is recommended that the listed alternative pipe arch sizes conform to those sizes shown for corrugated steel pipe arches shown on Table 856.3D. The designer should verify the availability of reinforced concrete pipe arches. If reinforced concrete pipe arches are not available, oval shaped reinforced concrete pipe of a size necessary to meet the hydraulic requirements may be used as an alternative.

CHAPTER 870 CHANNEL AND SHORE PROTECTION - EROSION CONTROL

Topic 871 - General

Index 871.1 - Introduction

Highways are often attracted to parallel locations along streams, coastal zones and lake shores. These locations are under attack from the action of waves and flowing water that may require protective measures.

Channel and shore protection can be a major element in the design, construction, and maintenance of highways. This section deals with procedures, methods, devices, and materials commonly used to mitigate the damaging effects of flowing water and wave action on highway facilities and adjacent properties. Potential sites for such measures should be reviewed in conjunction with other features of the project such as long and short term protection of downstream water quality, aesthetic compatibility with surrounding environment, and ability of the newly created ecological system to survive with minimal maintenance. See Index 110.2 for further information on water quality and environmental concerns related to erosion control.

Refer to Topic 874 for definition of drainage terms.

871.2 Design Philosophy

In each district there should be a designer or advisor, usually the District Hydraulic Engineer, knowledgeable in the application of bank protection principles and the performance of existing works. Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from involvement with Design, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802).

There are a number of ways to deal with the problem of wave action and stream flow.

- The simplest way and generally the surest of success and permanence, is to locate the roadway away from the erosive forces. This is not always feasible or economical, but should be the first consideration. Locating the roadway to higher ground or solid support should never be overlooked, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for protection at other points of attack.
- The most commonly used method is to armor the embankment with a more resistant material like rock slope protection. The type of material to be used for the protection is discussed under Topic 872.
- A third method is to reduce the force of the attacking water. This is often done by means of retards, permeable jetties and various plantings such as willows. Plantings once established not only reduce stream velocity near the bank during heavy flows, but their roots add structure to the bank material.
- Another method is to direct the attacking water away from the embankment. In the case of wave attack, additional beach may be created between the embankment and the water by means of groins and sills which trap littoral drift or hold imported sand. In the case of stream attack, a new channel can be created or the stream can be diverted away from the embankment by the use of jetties, baffles, deflectors, groins or spurs.

Combinations of the above four methods may be used. Even protective works destroyed in floods have proven to be effective and cost efficient in minimizing damage to highways.

Design of protective features should be governed by the importance of the facility and appropriate design principles. Some of the factors which should be considered are:

- *Roughness*. Revetments generally are less resistant to flow than the natural channel bank. Channel roughness can be significantly reduced if a rocky vegetated bank is denuded of trees and rock outcrops. When a rough natural bank is replaced by a smooth revetment, the current is accelerated, increasing its power to erode, especially along the toe and downstream end of the revetment. Except in narrowed channels, protective elements should approximate natural roughness. Retards, baffles and jetties can simulate the effect of trees and boulders along natural banks and in overflow channels.

- *Undercutting*. Particular attention must be paid to protecting the toe of revetments against undercutting caused by the accelerated current along smoothed banks, since this is the most common cause of bank failure.

- *Standardization*. Standardization should be a guide but not a restriction in designing the elements and connections of protective structures.

- *Expendability*. The primary objectives of the design are the safety of the traveling public and the security of the highway, not security of the protective structure. Less costly replaceable protection may be more economical than expensive permanent structures.

- *Dependability*. An expensive structure is warranted primarily where highways carry high traffic volumes, where no detour is available, or where roadway replacement is very expensive.

- *Longevity*. Short-lived structures or materials may be economical for temporary situations. Expensive revetments should not be placed on banks likely to be buried in widened embankments, nor on banks attacked by transient meander of mature streams.

- *Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity of stone is a major factor in shore protection and the specified minimum should not be lowered without increasing the mass of stones. For example, 10 percent decrease in specific gravity requires a 55 percent increase in mass (say from a 9 ton stone to a 14 ton stone) for equivalent protection.

- *Selection*. Selection of class and type of protection should be guided by the intended function of the installation.

- *Limits*. Horizontal and vertical limits of protection should be carefully designed. The bottom limit should be secure against toe scour. The top limit should not arbitrarily be at high-water mark, but above it if overtopping would cause excessive damage and below it if floods move slowly along the upper bank. The end limits should reach and conform to durable natural features or be secure with respect to design parameters.

871.3 Selected References

Hydraulic and drainage related publications are listed by source under Topic 807. References specifically related to slope protection measures are repeated here for convenience.

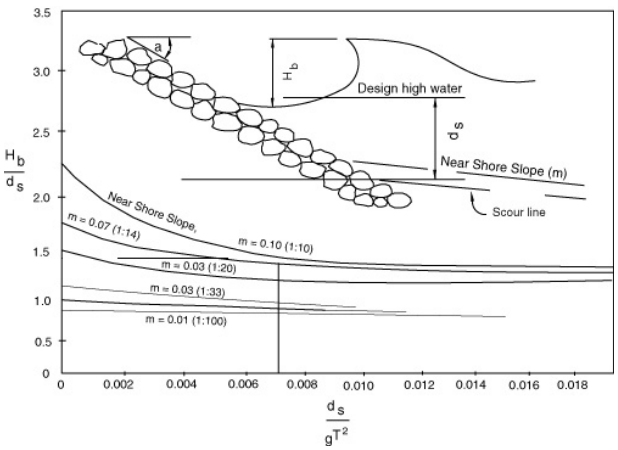
(a) FHWA Hydraulic Engineering Circulars (HEC) -- The following five circulars were developed to assist the designer in using various types of slope protection and channel linings:

- HEC 11, Design of Riprap Revetment (2000)
- HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006)
- HEC 15, Design of Roadside Channels with Flexible Linings (2005).
- HEC 18, Evaluating Scour at Bridges (2001)
- HEC 20, Stream Stability at Highway Structures (2001)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
- HEC 25, Highways in the Coastal Environment (2008)

(b) FHWA Hydraulic Design Series (HDS) No. 6, River Engineering for Highway Encroachments (2001) -- A comprehensive treatise of natural and man-made impacts and responses on the river environment, sediment transport, bed and bank stabilization, and countermeasures.

(c) AASHTO Highway Drainage Guidelines -- General guidelines for good erosion control

Figure 873.2C
Design Breaker Wave



Example
By using hindcast methods, the significant wave height (H_s) has been estimated at 4 feet with a 3 second period. Find the design wave height (H_d) for the slope protection if the depth of water (d) is only 2 feet and the nearshore slope (m) is 1:10.

Solution

$$\frac{d_s}{gT^2} = \frac{2\ \text{ft}}{(32.2\ \text{ft/s}^2) \times (3\ \text{sec})^2} = 0.007$$

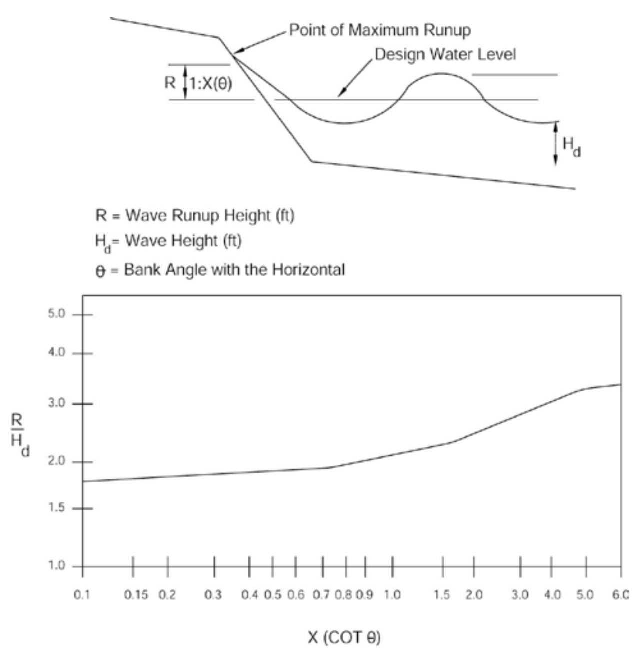
From Graph) - $H_b/d_s = 1.4$
$H_b = 2 \times 1.4 = 2.8$ ft

Answer
Since the maximum breaker wave height, H_b, is smaller than the significant deepwater wave height, H_s, the design wave height H_d is 2.8 feet.
T = Wave Period (SPM)

Procedures for estimating wave run-up for smooth surfaces (e.g., concrete paved slopes) and for vertical and curved face walls are contained in the U.S. Army Corps of Engineers, Shore Protection Manual, 1984. See Figure 873.2D for estimating wave run-up on smooth slopes for wave heights of 2 feet or less.

In protected bays and estuaries, waves generated by recreational or commercial boat traffic and other watercraft may dominate the design over wind generated waves. Direct observation and measurements during high tidal cycles may provide the designer the most useful tool for establishing wave run-up for these situations.

Figure 873.2D
Wave Run-up on Smooth Impermeable Slope



(c) Littoral Processes. Littoral processes result from the interaction of winds, waves, currents, tides, and the availability of sediment. The rates at which sediment is supplied to and removed from the shore may cause excessive accretion or erosion that can effect the structural integrity of shore protection structures or functional usefulness of a beach. The aim of good shore protection design is to maintain a stable shoreline where the volume of sediment supplied to the shore balances that which is removed.

Designers interested in a more complete discussion on littoral processes should consult the U.S. Army Corps of

Engineers' Coastal Engineering Manual (CEM) – Part III.

873.3 Armor Protection

(1) *General.* Armor is the artificial surfacing of bed, banks, shore or embankment to resist erosion or scour. Armor devices can be flexible (self adjusting) or rigid.

Hard armoring of stream banks and shorelines, primarily with rock slope protection (RSP), has been the most common means of providing long-term protection for transportation facilities, and most importantly, the traveling public. With many years of use, dozens of formal studies and thousands of constructed sites, RSP is the armor type for which there exists the most quantifiable data on performance, constructability, maintainability and durability, and for which there exist several nationally recognized design methods.

Due to the above factors, RSP is the general standard against which other forms of armoring are compared. The results of internal research led to the publication of Report No. FHWA-CA-TL-95-10, "California Bank and Shore Rock Slope Protection Design". Within that report, the methodology for RSP design adopted as the Departmental standard, is the California Bank and Shore, (CABS), layered design. The full report is available at the following website:

http://www.dot.ca.gov/hq/oppd/hydrology/hydroidx.htm.

This design method, which is applied with slight variation to ocean and lake shores vs. stream banks, and is also followed for concreted RSP designs, is the only protection method as of this writing that has been formally adopted by the Caltrans Bank and Shore Protection Committee. Section 72 of the Standard Specifications provides all construction and material specifications for RSP designs. While standards (i.e., Standard Plans, Standard Specifications and/or SSP's) do exist for some other products discussed in this Chapter (most notably for gabions, but also for certain rolled or mat-style erosion control products), their primary application is for relatively flat slope or shallow ditch erosion control (gabions are also used as an earth retaining structure, see Topic 210 for more details).

Other armor types listed below and described throughout this Chapter are viable and may be used, upon approval of the Headquarters Hydraulic Engineer or Caltrans Bank and Shore Protection Committee, where conditions warrant. Although the additional step of headquarters approval of these non-standard designs is required, designers are encouraged to consider alternative designs, particularly those that incorporate vegetation or products naturally present in stream environments. The District Landscape Architect can provide design assistance together with specifications and details for the vegetative portion of this work.

(a) Flexible Types.

- Rock slope protection.
- Broken concrete slope protection.
- Broken concrete, uncoursed.
- Gabions, Standard Plan D100A and D100B.
- Precast concrete articulated blocks.
- Rock filled cellular mats.

(b) Rigid Types.

- Concreted-rock slope protection.
- Sacked concrete slope protection.
- Concrete slope protection.
- Concrete filled fabric slope protection.
- Air-blown mortar.
- Soil cement slope protection.

(c) Other Armor types:

(1) Channel Liners and Vegetation. Temporary channel lining can be used to promote vegetative growth in a drainage way or as protection prior to the placement of permanent armoring. This type of lining is used where an ordinary seeding and mulch application would not be expected to withstand the force of the channel flow. In addition to the following, other suitable products

of natural or synthetic materials are available that may be used as temporary or permanent channel liners.

- Excelsior
- Jute
- Paper mats
- Fiberglass roving
- Geosynthetic mats or cells
- Pre-cast concrete blocks with open cells
- Brush layering
- Rock riprap in sizes smaller than backing No. 3

(2) Bulkheads. The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Crib walls
- Sheet piling
- Sea Walls

(d) General Design Criteria. In selecting the type of flexible or rigid armor protection to use the following characteristics are important design considerations.

(1) The lower limit, or toe, of armor should be below anticipated scour or on bedrock. If for any reason this is not economically feasible, a reasonable degree of security can be obtained by placement of additional quantities of heavy rock at the toe which can settle vertically as scour occurs.

(2) In the case of slope paving or any expensive revetment which might be seriously damaged by overtopping and subsequent erosion of underlying embankment, extension above design high water may be warranted. The usual limit of extension for streambank protection above design high water is 1 foot to 2 feet in unconstricted reaches and 2 feet to 3 feet in constricted reaches.

(3) The upstream terminal can be determined best by observation of existing conditions and/or by measuring velocities along the bank.

The terminal should be located to conform to outcroppings of erosion-resistant materials, trees, shrubs or other indications of stability.

In general, the upstream terminal on bends in the stream will be some distance upstream from the point of impingement or the beginning of curve where the effect of erosion is no longer damaging.

(4) When possible the downstream terminal should be made downstream from the end of the curve and against outcroppings, erosion-resistant materials, or returned securely into the bank so as to prevent erosion by eddy currents and velocity changes occurring in the transition length.

(5) The encroachment of embankment into the stream channel must be considered with respect to its effect on the conveyance of the stream and possible damaging effect on properties upstream due to backwater and downstream due to increased stream velocity or redirected stream flow.

(6) A smooth surface will generally accelerate velocity along the bank, requiring additional treatment (e.g., extended transition, cut-off wall, etc.) at the downstream terminal. Rougher surfaces tend to keep the thread of the stream toward the center of the channel.

(7) Heavy-duty armor used in exposures along the ocean shore may be influenced or dictated by economics, or the feasibility of handling heavy individual units.

(2) *Flexible Revetments.*

(a) Streambank Rock Slope Protection.

(1) General Features. This kind of protection, commonly called riprap, consists of rock courses placed upon the embankment or the natural slope along a stream. Rock, as a slope protection material, has a number of desirable features which have led to its widespread application.

It is usually the most economical type of revetment where stones of sufficient size and quality are available, it also has the following advantages:

- It is flexible and is not impaired nor weakened by slight movement of the embankment resulting from settlement or other minor adjustments.
- Local damage or loss is easily repaired by the addition of similar sized rock where required.
- Construction is not complicated and special equipment or construction practices are not usually necessary. (Note that Method A placement of very large rock may require large cranes or equipment with special lifting capabilities).
- Appearance is natural, and usually acceptable in recreational and scenic areas.
- If exposed to fresh water, vegetation may be induced to grow through the rocks adding structural value to the embankment material and restoring natural roughness.
- Additional thickness (i.e., mounded toe design) can be provided at the toe to offset possible scour when it is not feasible to found it upon bedrock or below anticipated scour.
- Wave run-up is less than with smooth types (See Figure 873.2D).
- It is salvageable, may be stockpiled and reused if necessary.

In designing the rock slope protection for a given embankment the following determinations are to be made for the typical section.

- Depth at which the stones are founded (bottom of toe trench).
- Elevation at the top of protection.
- Thickness of protection.
- Need for geotextile and backing material.
- Face slope.

(a) Placement -- Two different methods of placement for rock slope protection are allowed under Section 72 of the Standard Specifications: Placement under Method A requires considerable care, judgment, and precision and is consequently more expensive than Method B. Method A should be specified primarily where large rock is required, but also for relatively steeper slopes.

Under some circumstances the costs of placing rock slope protection with refinement are not justified and Method B placement can be specified. To compensate for a partial loss and assure stability and a reasonably secure protection, the thickness is increased over the more precise Method A by 25 percent.

(b) Foundation Treatment -- The foundation excavation must afford a stable base on bedrock or extend below anticipated scour.

Terminals of revetments are often destroyed by eddy currents and other turbulence because of nonconformance with natural banks. Terminals should be secured by transitions to stable bank formations, or the end of the revetment should be reinforced by returns of thickened edges.

While a significant amount of research is currently being conducted, few methods exist for estimating scour along stream banks. One of the few is the method contained in the CHANLPRO Program developed by the U.S. Army Corps of Engineers. Based on the flume studies at the Corps' Waterways Experiment Station, the program is primarily used by the Corps for RSP designs on streams with 2 percent or lesser gradients, but contains an option for scour depth estimates in bends for sand channels. CHANLPRO is available at the following USACE website: http://chl.erdc.usace.army.mil/CHL.aspx?p=s&a=Software;3 along with a user guide containing equations, charts, assumptions and limitations to the method and example problems.

(c) Embankment Considerations -- Embankment material is not normally carried out over the rock slope protection so that the rock becomes part of the fill. With this type of construction fill material can filter down through the voids of the large stones and that portion of the fill above the rocks could be lost. If it is necessary to carry embankment material out over the rock slope protection a geotextile is required to prevent the loses of fill material.

The embankment fill slope is usually determined from other considerations such as the angle of repose for embankment material, or the normal 1V:4H specified for high-standard roads. If the necessary size of rock for the given exposure is not locally available, consideration should be given to flattening of the embankment slope to allow a smaller size stone, or substitution of other types of protection. On high embankments, alternate sections on several slopes should be compared, practically and economically; flatter slopes require smaller stones in thinner sections, but at the expense of longer slopes, a lower toe elevation, increased embankment, and perhaps additional right of way.

Where the roadway alignment is fixed, slope flattening will often increase embankment encroachment into the stream. When such an encroachment is environmentally or technically undesirable, the designer should consider various vertical, or near vertical, wall type alternatives to provide adequate stream width, allowing natural channel migration and the opportunity for enhancing habitat.

(d) Rock Slope Protection Fabric and Inner Layers of Rock -- The layered method of designing RSP installations was developed prior to widespread availability of the rock slope protection fabrics which are described in Standard Specification Section 88. The RSP fabric and multiple layers of rock ensure that fine soil particles do not migrate through the RSP due to hydrostatic forces and, thus, eliminate the potential for bank failure. The use of RSP fabric provides an inexpensive layer of protection retaining embankment fines in lieu of placing backing No. 3 or similar small, well graded materials. See Index 873.3(2)(a)(1)(e) "Gravel Filter."

Under special circumstances, the designer may consider allowing holes to be cut in the RSP fabric, generally to facilitate more rapid/extensive rooting of woody vegetation through the RSP revetment. This practice is only necessary for deeply rooted plant species. Holes in RSP fabric should not be cut below the stage of the 2-year return period event. The District Hydraulic Unit should be consulted for advice prior to any determination to cut or otherwise modify standard installation of RSP fabric.

Additionally, stronger and heavier RSP fabrics than those listed in the Standard Specifications are manufactured. They are used in special designs for larger than standard RSP sizes, or emergency installations where placement of the layered design is not feasible and large RSP must be placed directly on the fabric. These heavy weight fabrics have unit weights of up to 16 ounces per square yard. Contact the Headquarters Hydraulic Engineer for assistance regarding usage applications of heavy weight RSP fabrics.

(e) Gravel Filter -- Generally RSP fabric should always be used unless there is a permit requirement for establishment of vegetation that precludes the placement of fabric due to inadequate root penetration. Where RSP fabric cannot be placed, such as in stream environments where CA Fish & Game and NOAA Fisheries strongly discourage the use of RSP Fabric, a gravel filter is usually necessary with most native soil conditions to stop fines from bleeding through the typical RSP classes.

When a gravel filter is to be placed, the designer is advised to work with the District Materials Office to get a recommendation for the necessary gradation to work effectively with both the native backfill and the base layer of the RSP that is being placed. Among the methods available for designing the gravel filter are the Terzaghi method, developed exclusively for situations where the native backfill is sand, and the Cisten-Ziems method, which is often used for a broad variety of soil types. Where streambanks must be significantly rebuilt and reconfigured with imported material before RSP placement, the designer must ensure that the imported material will not bleed through the designed gravel filter.

(2) Streambank Protection Design. In the lower reaches of larger rivers wave action resulting from navigation or wind blowing over long reaches may be much more serious than velocity. A 2 foot wave, for example, is more damaging than direct impingement of a current flowing at 10 feet per second.

Well designed streambank rock slope protection should:

- Assure stability and compatibility of the protected bank as an integral part of the channel as a whole.
- Connect to natural bank, bridge abutments or adjoining improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.
- Eliminate or ease local embayments and capes so as to streamline the protected bank.
- Consider the effects of backwater above constrictions, superelevations on bends, as well as tolerance of occasional overtopping.
- Not be placed on a slope steeper than 1.5H:1V. Flatter slopes (see Figure 873.3A) use lighter stones in a thinner section and encourage overgrowth of vegetation, but may

not be permissible in narrow channels.

- Use stone of adequate weight to resist erosion, derived from Figure 873.3A.
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric and multiple layers of backing should be used.
- Rest on a good foundation on bedrock or extend below the depth of probable scour. If questionable, use heavy bed stones and provide a wide base section with a reserve of material to slough into local scour holes (i.e., mounded toe).
- Reinforce critical zones on outer bends subject to impinging flow, using heavier stones, thicker section, and deeper toe.
- Be constructed in two or more layers of rock sizes, with progressively smaller rock toward native bank to prohibit loss of soil fines.
- Be constructed of rock of such shape as to form a stable protection structure of the required section. Rounded boulders or cobbles must not be used on prepared ground surfaces having slopes steeper than 2.5H:1V

(a) Stone Size -- Where stream velocity governs, rock size may be estimated by using the nomogragh, Figure 873.3A.

The nomograph is derived from the following formula:

$$W = \frac{0.00002V^6 sg_r \csc^3(\beta - \alpha)}{(sg_r - 1)^3}$$

Where:

sg_r = specific gravity of stones

α = angle of face slope from the horizontal

β = 70 for broken rock, a constant

W = weight of minimum stable stone in lbs

V = 2/3 average stream velocity, fps (flow parallel to bank) or 4/3 average stream velocity, fps (flow impinging on bank)

Where wave action is dominant, design of rock slope protection should proceed as described for shore protection.

(b) Design Height -- The top of rock slope protection along a stream bank should be carried to the elevation of the design high water plus some allowance for freeboard. The flood stage elevation adopted for design may be based on an empirically derived frequency of recurrence (probability of exceedance) or historic high water marks. This stage may be exceeded during infrequent floods, usually with little or no damage to the upper slope.

Design high water should not be based on an arbitrary storm frequency alone, but should consider the cost of carrying the protection to this height, the probable duration and damage if overtopped, and the importance of the facility.

When determining freeboard, or the height above design high water from which the RSP is to extend, one should consider: the size and nature of debris in the flow; the

Figure 873.3A

Nomograph of Stream-Bank Rock Slope Protection

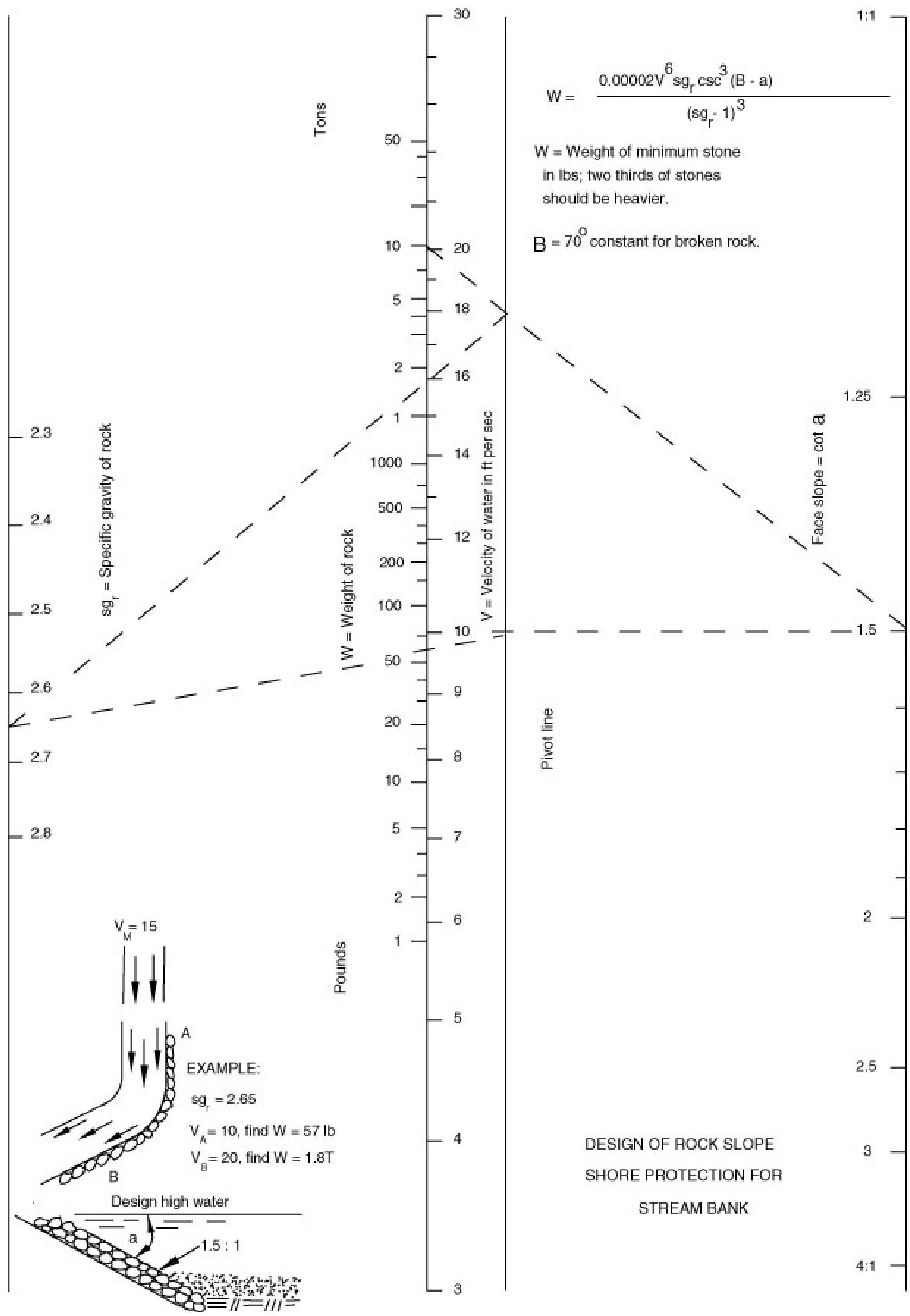
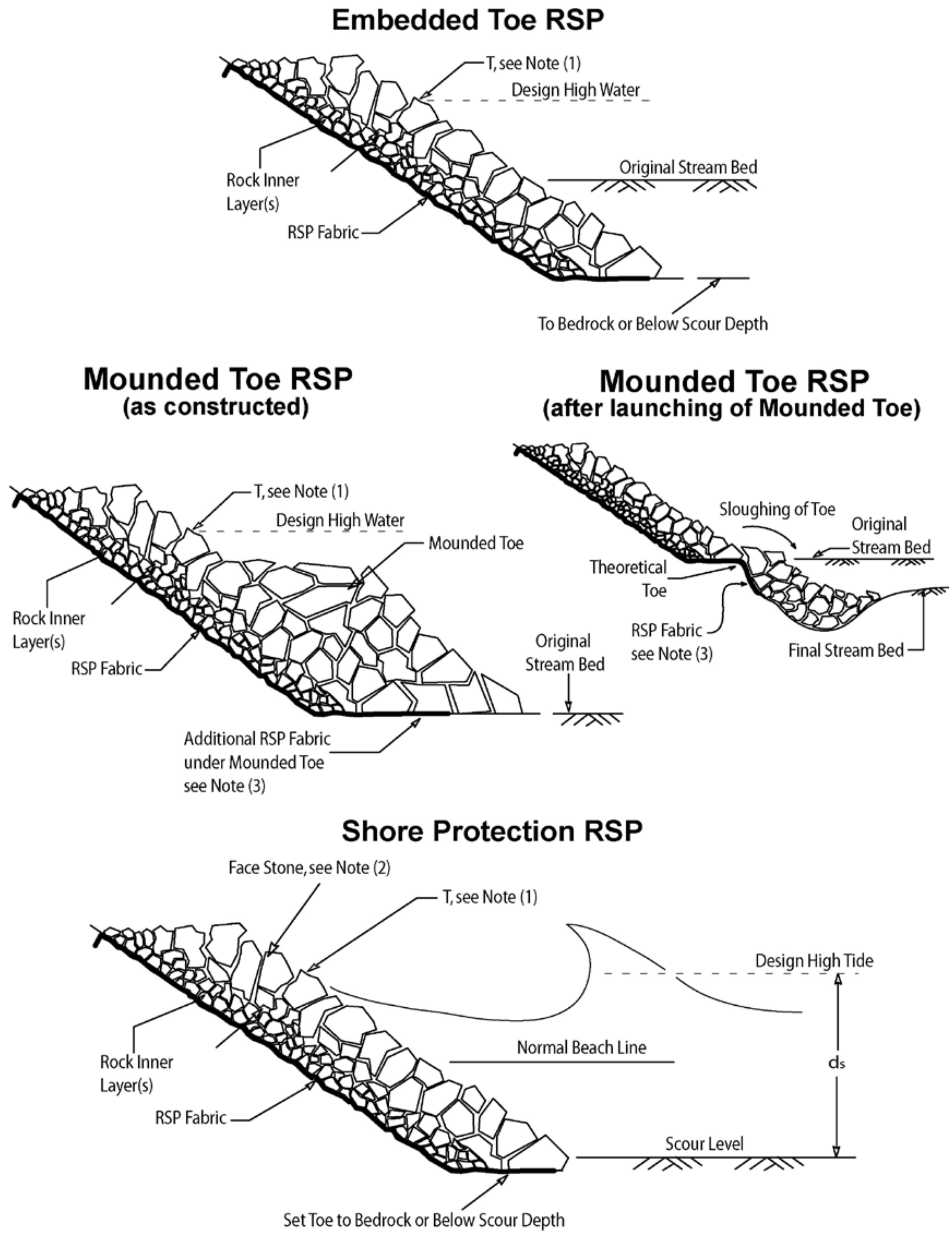


Figure 873.3C
Rock Slope Protection



Notes:

(1) Thickness "T" from Table 873.3 C.
(2) Face stone is determined from Figure 873.3G.
(3) RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

resulting potential for damage to the bank, the potential for streambed aggradation; and the confidence in data used to estimate design highwater. Freeboard may also be affected by regulatory or local agency requirements. Freeboard may be more generous along freeways, on bottleneck routes, on the outside bends of channels, or around critical bridges.

Design high water should be adjusted to the site based on sound engineering judgement.

Design Example -- The following example reflects the CABS method for designing RSP as described in Report No. FHWA – CA – TL – 95 – 10, as well as identify some of the considerations and technical principles that the designer must address to complete the installation design. These same considerations and principles apply to concreted RSP as well as RSP placed on beaches and shores (which are covered later), and therefore, separate examples for those designs are not provided. The designer is encouraged to review the entire report referenced above, available on the Division of Design website, for a comprehensive discussion of the basis of the CABS method and RSP design considerations.The following example assumes that the designer has conducted the appropriate site assessments and resulting calculations to establish average stream velocity, estimated depth of scour, stream alignment (i.e., parallel or impinging flow), length of stream bank to be protected and locations of natural hard points (e.g., rock outcroppings). Field reviews and discussions with maintenance staff familiar with the site are critical to the success of the design.

Given for example:

- Average stream velocity for design event – 16 feet per second
- Estimated scour depth – 5.5 feet
- Length of bank requiring protection – 550 feet
- Bank slope – 1.5:1
- Specific gravity of rock used for RSP – 2.65 (based on data from local quarry)
- Embankment is on outside of stream bend

1) Calculate minimum rock mass for outer layer:

$$W = \frac{(0.00002)\left(16 \times \frac{4}{3}\right)^6 (2.65)}{(2.65-1)^3 \sin^3(70-33.69)}$$

W = 5,350 lb

W = 2.67 ton = 2.43 tonne

NOTES:

For ease of computation with hand held calculators, cosecant has been converted to 1/sine.)

2) Select gradation for outer layer.

   a) From minimum calculated rock weight of 2.67 tons in the example, select the rock weight from the left-side column tables in Standard Specification Section 72-2.02 that represents the standard rock weight just larger than the calculated weight. For ease, the Standard Specification tables are combined and reprinted in Table 873.3A.

Table 873.3A
Guide for Determining RSP-Class of Outside Layer

Standard Rock Sizes	GRADING OF ROCK SLOPE PROTECTION PERCENTAGE LARGER THAN											
	RSP-Classes [1]											
	Method A Placement					Method B Placement						
	RSP-Classes other than Backing									Backing No.		
	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1	2	3
16 ton	0-5											
8 ton	50-100	0-5										
4 ton	95-100	50-100	0-5									
2 ton		95-100	50-100	0-5		0-5						
1 ton			95-100	50-100	0-5	50-100	0-5					
1/2 ton				95-100	50-100	-----	50-100	0-5				
1/4 ton					95-100	95-100	-----	50-100	0-5			
200 lb							95-100	-----	50-100	0-5		
75 lb								95-100	-----	50-100	0-5	
25 lb									95-100	90-100	25-75	0-5
5 lb											90-100	25-75
1 lb												90-100

[1] “Facing” has same gradation as “Backing No. 1”. To conserve space “Facing” is not shown.

The next larger rock mass above 2.67 ton is 4 ton. RSP this large is only to be installed using Method A placement techniques (i.e., individual rock placement, no end dumping). From this value, move horizontally across the gradation ranges to the "50-100" entry. From here, move vertically upward to select the design gradation, or RSP Class. In this instance the name of the RSP class is 4 T.

(b) Generally, this will represent the design outer RSP layer. However, the designer must assess this value against the site conditions observed during the field review and in conjunction with site history and projected future conditions prior to finalizing the selection. For the purposes of this example, we will assume this design gradation (i.e., 4 T RSP class) is appropriate.

3) Determine RSP Layers. As previously discussed, properly designed RSP revetments are comprised of multiple layers of progressively smaller rock gradations progressing from the large sized rocks of the outer layer to the native soil or constructed embankment. Where the outer layer is composed of relatively small rock only a single inner layer may be needed. For a large rock outer layer as many as three inner layers may be required.

For this example, the outer RSP layer is 4 T. From Table 873.3B, there are two options for the inner layers. The reason for multiple options for the larger RSP gradation classes is to allow the designer to better select RSP that is available from local quarry sources. Either set of layered designs is acceptable. The designer should contact rock producers in proximity to the project site to obtain price quotes for the different alternatives.

This information may also be available from the District Materials Engineer. For the purposes of this example, we will select the layered design of: 4 T, 1 T, ¼ T, Backing No. 2 and Class 10 RSP Fabric.

4) Determine Thickness of Revetment. RSP layers are composed of rock classes shown in Table 873.3A. Each layer is at least 1.5 times the diameter of the median sized rock (D_{50}) in the gradation in order to prevent the smaller rocks in the lower layers from migrating.

Table 873.3C provides the required thickness for the various RSP gradations and types of placement (Method A or Method B). Method B placement requires an increase in thickness to account for the looser rock contact and difficulty in controlling layer thickness inherent in end dumping of rock.

Based on the table values, the total thickness of the design in our example (measured normal to the slope) is:

Table 873.3B

California Layered RSP

Outsider Layer RSP-Class *	Inner Layers RSP-Class *	Backing Class No. *	RSP-Fabric Class **
8 T	2 T over ½ T	1	10
8 T	1 T over ¼ T	1 or 2	10
4 T	½ T	1	10
4 T	1 T over ¼ T	1 or 2	10
2 T	½ T	1	10
2 T	¼ T	1 or 2	10
1 T	Light	None	8
1 T	¼ T	1 or 2	8
½ T	None	1	8
¼ T	None	1 or 2	8
Light	None	None	8
Backing No.1 ***	None	None	8

* Rock grading and quality requirements per Standard Specifications.

** RSP-fabric Type of geotextile and quality requirements per Section 88 Rock Slope Protection Fabric of the Standard Specifications. Class 8 RSP-fabric has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric.

*** “Facing” RSP-Class has same gradation as Backing No. 1.

Table 873.3 C

Minimum Layer Thickness

RSP-Class Layer	Method of Placement	Minimum Thickness
8 T	A	8.5 ft
4 T	A	6.8 ft
2 T	A	5.4 ft
1 T	A	4.3 ft
½ T	A	3.4 ft
1 T	B	5.4 ft
½ T	B	4.3 ft
¼ T	B	3.3 ft
Light	B	2.5 ft
Facing	B	1.8 ft
Backing No. 1	B	1.8 ft
Backing No. 2	B	1.25 ft
Backing No. 3	B	0.75 ft

4 T Layer = 6.8 ft

1 T Layer = 4.3 ft

¼ T Layer = 3.3 ft

Backing No. 2 Layer = 1.25 ft

RSP Fabric = Effectively

\+ 0.0 ft

Total = 15.35 ft

5) Assess Stream Impact Due to Revetment. In some cases, the thickness of the completed RSP revetment creates a narrowing of the available stream channel width, to the extent that stream velocity or stage at the design event is increased to undesirable levels, or the opposite bank becomes susceptible to attack. In these cases, the bank upon which the RSP is to be placed

must be excavated such that the constructed face of the revetment is flush with the original embankment.

6) Exterior Edges of Revetment. The completed design must be compatible with existing and future conditions. Freeboard and top edge of revetments were covered in Index 873.3(2)(a)(2)(b) "Design Height." For depth of toe, the estimated scour was given as 5.5 feet. This is the minimum toe depth to be considered. Again, based on site conditions and discussions with maintenance staff and others, determine if any long-term conditions need to be addressed. These could include streambed degradation due to local aggregate mining or headcutting. Regardless of the condition, the toe must be founded below the lowest anticipated elevation that could become exposed over the service life of the embankment or roadway facility. As for the upstream and downstream ends, the given length of revetment is 500 feet. Again, this will typically be a minimum, as the designer should seek natural rock outcroppings, areas of quiescent stream flow, or other inherently stable bank segments to end the RSP, see Figure 873.3D for example at ocean shore location.

(b) Rock Slope Shore Protection.

(1) General Features. Rock slope protection when used for shore protection, in addition to the general advantages listed previously for streambank rock slope protection, reduces wave runup as compared to smooth types of protection.

(a) Method A placement is normally specified for ocean shore protection since very large stone is typically needed. Rock mass for lake shores and protected bays are often based on the height of boat generated waves.

(b) Foundation treatment in shore protection may be controlled by tidal action as well as excavation difficulties and production may be limited to only two or three toe or foundation rocks per tide cycle. If toe rocks are not properly bedded, the subsequent vertical adjustment may be detrimental to the protection above. Even though rock is self-adjusting, the bearing of one rock to another may be lost. It is often necessary to construct the toe or foundation to an elevation approximating high tide in advance of embankment construction to prevent erosion of the embankment.

(2) Shore Protection Design.

(a) Stone Size -- For waves that are shoaling as they approach the protection the required stone size may be determined by Using Chart B, Figure 873.3G.

The nomograph is derived from the following formula:

$$W = \frac{0.003 d_B{}^3 sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

d_B = maximum depth in feet of water at toe of the rock slope protection, see Figure 873.3C

sg_r = specific gravity of stones

sg_w = specific gravity of water (sea water = 1.0265)

α = angle of face slope from the horizontal

β = 70 for broken rock, a constant

W = weight of minimum stable stone in lbs

In general, d_B will be the difference between the elevation of the scour line at the toe and the maximum stillwater level. For ocean shore, d_S may be taken as the distance from the scour line to mean sea level plus one-half the maximum tidal range.

If the deep-water waves, see Figure 873.3D, reach the protection, the stone size may be determined by using Chart A, Figure 873.3G. The nomograph is derived from the following formula:

$$W = \frac{0.00231 H_d^{\,3} sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

H_d = design wave in feet, see Index 873.2

If in doubt whether waves generated by fetch and wind velocity will be of sufficient size to be affected by shoaling, use both charts and adopt the smaller value.

Figure 873.3D

RSP Lined Ocean Shore

RSP placed at site subject to deep water wave attack. Terminal end of RSP tied into natural rock outcropping.

(b) Dimensions -- Rock should be founded in a toe trench dug to hard rock or keyed into soft rock. If bedrock is not within reach, the toe should be carried below the estimated depth of probable scour. If the scour depth is questionable, additional thickness of rock may be placed at the toe which will adjust and provide deeper support. In determining the elevation of the scoured beach line the designer should observe conditions during the winter season, consult records, or ask persons who have a knowledge of past conditions.

Wave run-up is reduced by the rough surface of rock slope protection. In order that the wash will not top the rock, it should be carried up to an elevation of twice the maximum depth of water ($2d_S$) or to an elevation equal to the maximum depth of water plus the deep-water wave height ($d_S + H_d$), whichever is the *lower*. See Figure 873.3C.

Consideration should also be given to protecting the bank above the rock slope protection from splash and spray.

Design thickness of the protection should be based on the same procedures as used for streambanks. For typical conditions the thickness required for the various sizes are shown on Table 873.3B. Except for toes on questionable foundation, as explained above, additional thickness will not compensate for undersized stones. When properly constructed, the largest stones will be on the outside, and if the wave forces displace these, additional thickness will only add slightly to the time of failure. Shore revetments, particularly ocean shore locations, are often candidates for using a mounded toe design. Where it is not practical to excavate to bedrock or to the anticipated scour depth to set the revetment toe, an alternative treatment is to place additional rock (i.e., mound) of the same mass as the outer layer at the toe. The volume to be placed should be slightly greater than the amount that would have been needed to extend the toe to the estimated scour depth. See figure 873.3C for a depiction of a mounded toe installation.

As scour occurs at the toe of the revetment, this mounded rock will drop into the scour hole. It is important in mounded toe designs to require that excess RSP fabric be placed so that as the scour hole develops and rock begins to drop, the excess RSP fabric will "unfold" and also drop into place to limit loss of embankment.

(c) Gabions. Gabion revetments consist of rectangular wire mesh baskets filled with stone. See Standard Plan D100A and D100B for gabion basket details and the Standard Specifications for requirements.

Gabions are formed by filling commercially fabricated and preassembled wire baskets with rock. There are two types of gabions, wall type and mattress type. In wall type the empty cells are positioned and filled in place to form walls in a stepped fashion. Mattress type baskets are positioned on the slope and filled. Wall type revetment is not fully self adjusting but has some flexibility. The mattress type is very flexible. For some locations, gabions may be more aesthetically acceptable than rock riprap. Where larger stone sizes are not readily available and the flow does not abrade the wire baskets, they may also be more cost effective. However, caution is advised regarding in-stream placement of gabions, and some form of abrasion protection in the form of wooden planks or other facing will typically be necessary, see Figure 873.3E.

Figure 873.3E

Gabion Lined Streambank

Gabion wall with timber facing to protect wires from abrasive flow.

(d) Articulated Precast Concrete. This type of revetment consists of pre-cast concrete blocks which interlock with each other, are attached to each other, or butted together to form a continuous blanket or mat. A number of block designs are commercially available. They differ in shape and method of articulation, but share common features of flexibility and rapid installation. Most provide for establishment of vegetation within the revetment.

The permeable nature of these revetments permits free draining of the embankment and their flexibility allows the mat to adjust to minor changes in bank geometry. Pre-cast concrete block revetments may be economically justified where suitable rock for slope protection is not readily available. They are generally more aesthetically pleasing than other types of revetment, particularly after vegetation has become established.

Individual blocks are commonly joined together with steel cable or synthetic rope, to form articulated block mattresses. Pre-assembled in sections to fit the site, the mattresses can be used on slopes up to 2:1. They are anchored at the top of the revetment to secure the system against slippage.

Pre-cast block revetments that are formed by butting individual blocks end to end, with no physical connection, should not be used on slopes steeper than 3:1. An engineering fabric is normally used on the slope to prevent the erosion of the underlying embankment through the voids in the concrete blocks.

Refer to HEC-11, Design of Riprap Revetment, Section 6.2, and HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 4, for further discussion on the use of articulated concrete blocks.

(3) Rigid Revetments.

(a) Concreted-Rock Slope Protection.

(1) General Features. This type of revetment consists of rock slope protection with interior voids filled with PCC to form a monolithic armor. A typical section of this type of installation is shown in Figure 873.3F.

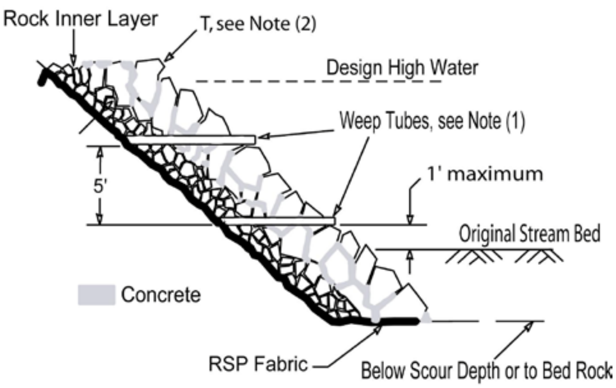
It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available.

(2) Design Concepts. Concreting of RSP is a common practice where availability of large stones is limited, or where there is a need to reduce the total thickness of a RSP revetment. Inclusion of the concrete, and the labor required to place it, makes concreted RSP installations more expensive per unit area than non-concreted installations.

Design procedures for concreted RSP revetments are similar to that of non-concreted RSP. Start by following the design example provided in Index 873.3(2)(a)(2)(c) to select a stable rock size for a non-concreted design based on the site conditions. This non-concreted rock size is divided by a factor of roughly four or five to arrive at the appropriate size outer layer rock for a concreted revetment. The factor is based on observations of previously constructed facilities and represents the typical sized pieces that stay together even after severe cracking (i.e., failed revetments will still usually have segments of four to five rocks holding together). As with the non-concreted design procedures, use the rock size

Figure 873.3F

Concreted-Rock Slope Protection



Notes:

(1) If needed to relieve hydrostatic pressure.

(2) Refer to Table 873.3 C for section thickness.

Dimensions and details should be modified as required.

derived from this calculation to enter Table 873.3A (i.e., round up to the next larger rock mass, which will represent the 50-100 percentage larger than gradation range) and then select the appropriate RSP Class. The thickness and rock sizing of the inner layers can be based on the reduced sizing of the outer layer rock. Note that as shown in Figure 873.3F, the inner layers of rock are not concreted.

As this type of protection is rigid without high strength, support by the embankment must be maintained. Slopes steeper than the angle of repose of the embankment are risky, but with rocks grouted in place, little is to be gained with slopes flatter than 1.5:1. Precautions to prevent undermining of embankment are particularly important, see Figure 873.3H. The concreted-rock must be founded on solid rock or below the depth of possible scour. Ends should be protected by tying into stable rock or forming smooth transitions with embankment subjected to lower velocities. As a precaution, cutoff stubs may be provided. If the embankment material is exposed at the top, freeboard is warranted to prevent overtopping.

The design intent is to place an adequate volume of concrete to tie the rock mass together, but leave the outer face roughened with enough rock projecting above the concrete to slow flow velocities to more closely approximate natural conditions.

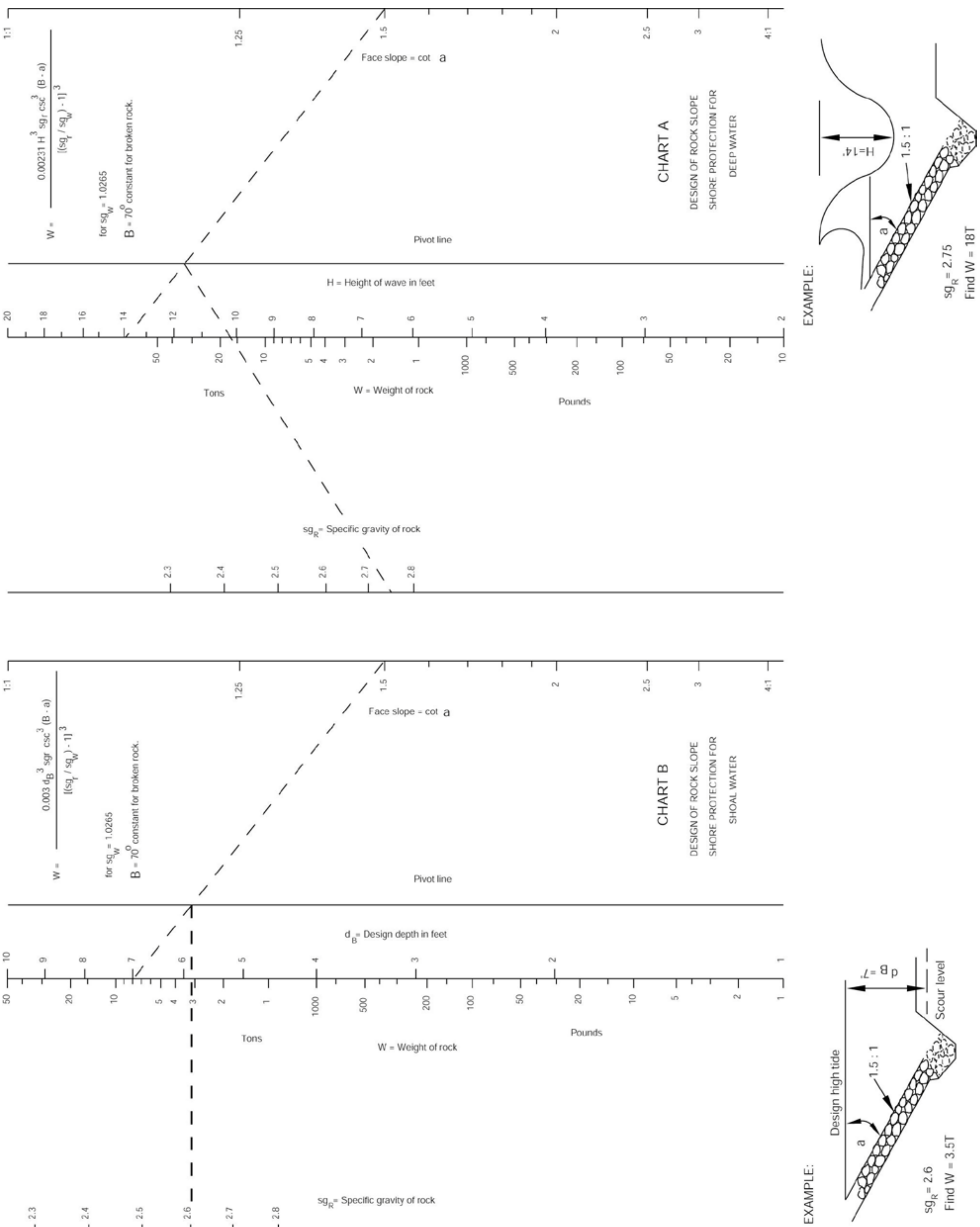
The volume of concrete required is based on filling roughly two-thirds of the void space of the outer rock layer, as shown in Figure 873.3F. The concrete is rodded or vibrated into place leaving the outer stones partially exposed. Void space for the various RSP gradations ranges from approximately 30 percent to 35 percent for Method A placed rock to 40 percent to 45 percent for Method B placed rock of the total volume placed.

Figure 873.3H

Toe Failure - Concreted RSP

Toe of concreted RSP that has been undermined.

Figure 873.3G
Nomographs For Design of Rock Slope Shore Protection



(2) Specifications. Quality specifications for rock used in concreted-rock slope protection are usually the same as for rock used in ordinary rock slope protection. However, as the rocks are protected by the concrete which surrounds them, specifications for specific gravity and hardness may be lowered if necessary. The concrete used to fill the voids is normally 1 inch maximum size aggregate minor concrete. Except for freeze-thaw testing of aggregates, which may be waived in the contract special provisions, the concrete should conform to the provisions of Standard Specification Section 90.

Size and grading of stone and concrete penetration depth are provided in Standard Specification Section 72.

(b) Sacked-Concrete Slope Protection. This method of protection consists of facing the embankment with sacks filled with concrete. It is expensive, but historically was a much used type of revetment. Much hand labor is required but it is simple to construct and adaptable to almost any embankment contour. Use of this method of slope protection is generally limited to replacement or repair of existing sacked concrete facilities, or for small, unique situations that lend themselves to hand-placed materials.

Tensile strength is low and as there is no flexibility, the installation must depend almost entirely upon the stability of the embankment for support and therefore should not be placed on face slopes much steeper than the angle of repose of the embankment material. Slopes steeper than 1:1 are rare; 1.5:1 is common. The flatter the slope, the less is the area of bond between sacks. From a construction standpoint it is not practical to increase the area of bond between sacks; therefore for slopes as flat as 2:1 all sacks should be laid as headers rather than stretchers.

Integrity of the revetment can be increased by embedding dowels in adjoining sacks to reinforce intersack bond. A No. 3 deformed bar driven through a top sack into the underlying sack while the concrete is still fresh is effective. At cold joints, the first course of sacks should be impaled on projecting bars that were driven into the last previously placed course. The extra strength may only be needed at the perimeter of the revetment.

Most failures of sacked concrete are a result of stream water eroding the embankment material from the bottom, the ends, or the top.

The bottom should be founded on bedrock or below the depth of possible scour.

If the ends are not tied into rock or other nonerosive material, cutoff returns are to be provided and if the protection is long, cutoff stubs are built at 30-foot intervals, in order to prevent or retard a progressive failure.

Protection should be high enough to preclude overtopping. If the roadway grade is subject to flooding and the shoulder material does not contain sufficient rock to prevent erosion from the top, then pavement should be carried over the top of the slope protection in order to prevent water entering from this direction.

Class 8 RSP fabric as described in Standard Specification Section 88 should be placed behind all sacked concrete revetments. For revetments over 4 feet in height, weep tubes should also be placed, see Figure 873.3F.

For good appearance, it is essential that the sacks be placed in horizontal courses. If the foundation is irregular, corrective work such as placement of entrenched concrete or sacked concrete is necessary to level up the foundation. Refer to HDS No. 6, Section 6.6.5, for further discussion on the use of sacked concrete slope protection.

(c) Concrete Slope Paving.

(1) General Features. This method of protection consists of paving the embankment with portland cement concrete. Slope paving is used only where flow is controlled and will not over-top the protection.

It is particularly adaptable to locations where high-velocity flow is not detrimental but desirable and the hydraulic efficiency of smooth surfaces is important. It has been used very little in shore protection. On a cubic feet basis the cost is high but as the thickness is generally only 3 inches to 6 inches, the cost on a basis of area covered will usually be less than for sacked-concrete slope protection. This is especially so when sufficiently large quantities are involved and alignment is such as to warrant the use of mass production equipment such as slip-form pavers.

Due to the rigidity of PCC slope paving, its foundation must be good and the embankment stable. Although reinforcement will enable it to bridge small settlements of the embankment face, even moderate movements could lead to cracking of the paving or failure. The toe must be on bedrock or extend below possible scour. When this is not feasible without costly underwater construction, rock or PCC grouted RSP have been used as a foundation. A better but much more expensive solution is to place the toe on a PCC wall or piles.

Every precaution must be taken to exclude stream water from pervious zones behind the slope paving. The light slabs will be lifted by comparatively small hydrostatic pressures, opening joints or cracks at other points in a series of progressive failures leading to extensive or complete failure.

Considering the severity of failure from bank erosion or hydrostatic pressure after overtopping, 1 foot to 2 feet of freeboard above design high water is recommended for this type of revetment. Refer to HEC-11, Design of Riprap Revetment, Section 6.4, for further discussion on the use of concrete slope paving. Table 873.3D gives channel lining thickness.

Table 873.3D

Channel Linings

Mean Velocity (ft/s)	Thickness of Lining (in)		Minimum Reinforcement
	Sides	Bottom	
Portland Cement Concrete or Air Blown Mortar			
< 10	3 – 3.5	3.5 – 4	6 x 6-W2.9 x W2.9 welded wire fabric
10 – 15	4 – 5	5 – 6	#4 Bars at 12 in. and 18 in. centers
15 or more	6 – 8	7 – 8	#3 Bars at 12 in. centers both ways

(4) *Bulkheads.* A bulkhead is a steep or vertical structure supporting a natural slope or constructed embankment. As bank and shore protection structures, bulkheads serve to secure the bank against erosion as well as retaining it against sliding. As a slope protection structure, revetment design principles are used, the only essential difference being the steepness of the face slope. As a retaining structure, conventional design methods for retaining walls, cribs and laterally loaded piles are used.

Bulkheads are usually expensive, but may be economically justified in special cases where valuable riparian property or improvements are involved and foundation conditions are not satisfactory for less expensive types of slope protection. They may be used for toe protection in combination with other revetment types of slope protection. Some other considerations that may justify the use of bulkheads include:

- Encroachment on a channel cannot be tolerated.
- Retreat of highway alignment is not viable.
- Right of Way is restricted.
- The force and direction of the stream can best be redirected by a vertical structure.

The foundation for bulkheads must be positive and all terminals secure against erosive forces. The length of the structure should be the minimum necessary, with transitions to other less expensive types of slope protection when possible. Eddy currents can be extremely damaging at the terminals and transitions. If overtopping of the bulkheads is anticipated, suitable protection should be provided.

Along a stream bank, using a bulkhead presumes a channel section so constricted as to prohibit use of a cheaper device on a natural slope. Velocity will be unnaturally high along the face of the bulkhead, which must have a fairly smooth surface to avoid compounding the restriction. The high velocity will increase the threat of scour at the toe and erosion at the downstream end. Allowance must be made for these threats in selecting the type of foundation, grade of footing, penetration of piling, transition, and anchorage at downstream end. Transitions at both ends may appropriately taper the width of channel and slope of the bank. Transition in roughness is desirable if attainable. Refer to HDS No. 6, Section 6.4.8, for further discussion on the use of bulkheads to prevent streambank erosion or failure.

Along a shore, use of a bulkhead presumes a steep lake or sea bed profile, such that revetment on a 1.5:1 or flatter slope would project into prohibitively deep water or permit intolerable wave runup. Such shores are generally rocky, offering good foundation on residual reefs, but historic destruction of the overlying formation attests to the hydraulic power of the sea to be resisted by an artificial replacement. The face of such a bulkhead must be designed to absorb or dissipate as much as practical the shock of these forces. Designers should consult the U.S. Army Corps of Engineers EM-1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads, for more complete information and details.

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

(b) Crib walls. Timber and concrete cribs can be used for bulkheads in locations where some flexibility is desirable or permissible. Metal cribs are limited to support of embankment and are not recommended for use as protection because of vulnerability to corrosion and abrasion.

The design of crib walls is essentially a determination of line, foundation grade, and height with special attention given to potential scour and possible loss of backfill at the base and along the toe. Design details for concrete crib walls are shown on Standard Plans C7A through C7G. Concrete crib walls used as bulkheads and exposed to salt water require special provisions specifying the use of coated rebars and special high density concrete. Recommendations from METS Corrosion Technology Branch should be requested.

Design details for timber crib walls of dimensioned lumber are shown on Standard Plans C9A and C9B. Timber cribs of logs, notched to interlock at the contacts, may also be used. All dimensioned lumber should be treated to resist decay.

(c) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 15 feet below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(5) *Vegetation*. Vegetation is the most natural method for stabilization of embankments and channel bank protection. Vegetation can be relatively easy to maintain, visually attractive and environmentally desirable. The root system forms a binding network that helps hold the soil. Grass and woody plants above ground provide resistance to the near bank water flow causing it to lose some of its erosive energy.

Erosion control and revegetation mats are flexible three-dimensional mats or nets of natural or synthetic material that protect soil and seeds against water erosion prior to establishment of vegetation. They permit vegetation growth through the web of the mat material and have been used as temporary channel linings where ordinary seeding and mulching techniques will not withstand erosive flow velocities. The designer should recognize that flow velocity estimates and a particular soils resistance to erosion are parameters that must be based on specific site conditions. Using arbitrarily selected values for design of vegetative slope protection without consultation with the District Hydraulic Unit and/or the District Landscape Architect Unit is not recommended. However, a suggested starting point of reference is Table 862.2 in which the resistance of various unprotected soil classifications to flow velocities are given. Under near ideal conditions, ordinary seeding and mulching methods cannot reasonably be expected to withstand sustained flow velocities above 4 feet per second. If velocities are in excess of 4 feet per second, a lining maybe needed, see Table 873.3E.

Temporary channel liners are used to establish vegetative growth in a drainage way or as slope protection prior to the placement of a permanent armoring. Some typical temporary channel liners are:

- Straw
- Excelsior
- Jute
- Woven paper

Vegetative and temporary channel liners are suitable for conditions of uniform flow and moderate shear stresses.

Permanent soil reinforcing mats and rock riprap may serve the dual purpose of temporary and permanent channel liner. Some typical permanent channel liners are:

- Gravel or cobble size riprap
- Fiberglass roving
- Geosynthetic mats
- Polyethelene cells or grids
- Gabion Mattresses

However, geosynthetics and plastic (polyethylene, polypropylene, polyamide, etc.) based mats must be installed in a fashion where there will be no potential for long-term sunlight exposure, as these products will degrade due to UV radiation.

Table 873.3E

Permissible Velocities for Flexible Channel Linings

Type of Lining [1]	Permissible Velocity (ft/s)	
	Intermittent Flow	Sustained Flow
Vegetation:		
Bermuda Grass, uncut	4.0	2.5
Bermuda Grass, mowed or Crab Grass, uncut	4.0	2.5
Riprap:		
Gravel, 1 in	3.0	2.0
Gravel, 2 in	3.5	2.5
Cobble, 3 in	5.0	4.0
Cobble, 6 in	7.5	6.5
Temporary:		
Woven Paper Net	4.5	3.5
Jute Net	5.0	4.0
Fiberglass Roving	5.5	4.5
Straw with Net	6.5	4.5
Curled Wood Mat	6.5	4.5
Synthetic Mat	10.5	7.5

NOTE:
(1) Ref. HEC-15

Composite designs are often used where there are sustained low flows of high to moderate velocities and intermediate high water flows of low to moderate velocities. Brush layering is a permanent type of erosion control technique that may also have application for channel protection, particularly as a composite design.

Additional design information on vegetation, and temporary and permanent channel liners is given in Chapter IV, HEC-15, Design of Roadside Channels and Flexible Linings.

873.4 Training Systems

(1) General. Training systems are structures, usually within a channel, that act as countermeasures to control the direction, velocity, or depth of flowing water. As shore protection, they control shoaling and scour by deflecting the strength of currents and waves.

The degree of permeability is among the most important properties of control structures. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce the strength of water velocity, currents or waves.

Training systems of the retard and permeable jetty types are similar in that they are usually extensive or multi-unit open structures like; piling, fencing, and unit frames. They are dissimilar in function and alignment, retards being parallel and groins oblique to the banks. The retard is a milder remedy than jetty construction.

(a) Retard Types. A retard is a bank protection structure designed to check riparian velocity and induce silting and accretion. They are usually placed parallel to the highway embankment or erodible banks of channels on stable gradients. Retards typically take the following forms of construction:

- Fencing - single or double lines
- Palisades - piles and netting
- Timber piling or pile bents
- Steel or timber jacks

Retards are applicable primarily on streams which meander to some extent within a mature valley. Typical uses include the following:

- Protection at the toe of highway embankments that encroach on a stream channel.
- Training and control to inhibit erosion upstream and downstream from stream crossings.
- Control of erosion redeposition of material where progressive embayments are creating a problem.

(1) Fence Type. Fence-type structures are used as retards, permeable or impermeable jetties, and as baffles. These structures can be constructed of various materials.

Fence type retards may be effective on smaller streams and areas subject to infrequent attack, such as overflow areas. Single and double rows of various types of fencing have been used. The principal difference between fence retards and ordinary wire fences is that the posts of retards must be driven sufficiently deep to avoid loss by scour.

Permeability can be varied in the design to fit the requirements of the location for single fences, the factor most readily varied is the pattern of the wire mesh. For multiple fences, the mesh pattern can be varied or the space between fences can be filled to any desired height. Making optimum use of local materials, this fill may be brush ballasted by rock, or rock alone.

(2) Piles and Palisades. Retards and jetties may be of single, double, or triple rows of piles with the outside or upstream row faced with wire mesh fencing material, boards or polymeric straps interwoven into a high-strength net. The facing adds to the retarding effect and may trap light brush or debris to supplement its purpose. This type

retard is particularly adapted to larger streams where the piles will remain in the water. The number of pile rows and amount of facing may be varied to control the deposition of material. In leveed rivers it is often desirable to discourage accretion so as to not constrict the channel but provide sufficient retarding effect to prevent loss of a light bank protection such as vegetation or light rock facing.

Typical design considerations include:

- If the stream carries heavy debris, the elevation of the top of the pile should be well below the high-water level in order that heavy objects such as logs will pass over the top during normal floods.
- Piles must have sufficient penetration to prevent loss from scour or impact by floating debris or both. This is especially important for the piles at the outer end of jetties. If scour is a problem, the pile may be protected by a layer of rock placed on the streambed. Piles should be long enough to penetrate below probable scour, with penetration of a least 15 feet in streams with sandy beds and velocities of 10 feet per second to 15 feet per second.
- Ends of the system should be joined to the bank in order to prevent parallel high-velocity flow between the retard and the bank. If the installation is long, additional bank connections may be placed at intervals.
- Facing material should be fastened to the upstream or channel side of the piling in order that the force of the water and impact of debris will not be entirely on the fasteners.

(3) Jacks and Tetrahedrons. Jacks and tetrahedrons are skeletal frames that can be used as retards or permeable jetties. Cables can be used to tie a number of similar units together in longitudinal alignment and for anchorage of key units to deadmen. Struts and wires are added to the basic frames to increase impedance to flow of water directly by their own resistance and indirectly by the debris they collect.

Both devices serve best in meandering streams which carry considerable bed load during flood stages. Impedance of the stream along the string of units will cause deposit of alluvium, especially at the crest and during the falling stage. Beds of such streams often scour on the rising stage, undercutting the units and causing their subsidence, often accompanied by rotation when one leg or side is undercut more than the other. Deposition of the falling stage usually restores the former bed, partially or completely burying the units. In that lowered and rotated position, they may still be completely effective in future floods.

Retards may be used alone or in combination with other types of slope protection. In combination with a lighter type of armor they may be more economical than a heavier type of protection. They can be used as toe protection for other types of slope protection where a good foundation is impractical because of high water or extreme depth of poor material.

Where new embankment is placed behind the retard consideration should be given to protecting the slope to inhibit erosion until the retard has had an opportunity to function. The slope protection used should promote the establishment of a natural cover, such as discussed under Index 873.3(5), Vegetation.

Retards on tangent reaches of narrow channels may, by slowing the velocity on one side, cause an increase in velocity, on the other. On wider reaches of a meandering stream they may, by slowing a rebounding high velocity thread, have a beneficial effect on the opposite bank. Where the prime purpose of the retard system is to reduce stream bank velocity to encourage deposition of material intended to alter the channel alignment the effect on adjacent property must be assessed. Where deposition of material is the primary function, the service life of the installation is dependent on the deposition rate and the ultimate establishment of a natural retard.

The length of a retard system should extend from a secure anchorage on the upstream end to anchorage on the downstream end beyond the area under direct attack. Since erosion often progresses downstream, this possibility should be considered in determining the planned length.

The top of a retard need not extend to the elevation of design high water. In major rivers and streams where drift is large and heavy it is essential that the retard be low enough to pass debris over the top during stages of high flow.

For further information on retards, refer to Section 6.4.4 of HDS No. 6.

(b) Jetty Types. A jetty is an elongated artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection or redirection of currents and waves. When used in stream environments, a common term used for these devices is spur dike.

This classification may be subdivided with respect to permeability. Impermeable jetties being used to deflect the stream and permeable jetties being used not only to deflect the stream but to permit some flow through the structure to minimize the formation of eddies immediately downstream. Most jetty installations are permeable structures.

Permeable jetties typically take the following forms of construction:

- Palisades -- piles and netting.
- Single and double rows of timber-braced piling.
- Steel or timber jacks.
- Precast concrete, interlocking shapes or hollow blocks.

Impermeable jetties typically take the following forms of construction:

- Guide and spur dikes, earth or rock.
- PCC grouted riprap dikes.
- Single and double lines of sheeting or sheet piling (steel, timber or concrete, framed and braced or on piling).
- Double fence, filled.
- Log or timber cribs, filled.

Impermeable jetties in the form of filled fences and cribs have been used with only limited success. Characteristic performance of these is the development of an eddy current immediately downstream which attacks the bank and often requires secondary protective measures.

Basic principles for permeable jetties are much the same as for retards, the important difference being that they deflect the flow in addition to encouraging deposition. The preceding comment on retards should be considered as related and applicable to jetties when qualified by this basic difference.

Permeable jetties are placed at an angle with the embankment and are more applicable in meandering streams for the purpose of directing or forcing the current away from the embankment, see Figure 873.4A. When the purpose is to deposit material and promote growth, the jetties are considered to have fulfilled their function and are expendable when this occurs.

Figure 873.4A

Thalweg Redirection Using Bendway Weirs

Bendway weirs in conjunction with rock slope protection.

They also encourage deposition of bed material and growth of vegetation. Retards build a narrow strip in front of the embankment, where as permeable jetties cover a wider area roughly limited by the envelope of the outer ends.

The relation between length and spacing of jetties should approximate unity as a general rule to assure complete entrapment and retention of material. The spacing can be increased if the resulting scalloped effect is not detrimental to the desired result. See HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 9 for additional information.

(c) Guide Dikes/Banks. Guide banks are appendages to the highway embankment at bridge abutments, see Figure 873.4B. They are smooth extensions of the fill slope on the upstream side. Approach embankments are frequently planned to project into wide floodplains, to attain an economic length of bridge. At these locations high water flows can cause damaging eddy currents that scour away abutment foundations and erode approach embankments. The purpose of guide dikes is twofold. The first is to align flow from a wide floodplain toward the bridge opening. The second is to move the damaging eddy currents from the approach roadway embankment to the upstream end of the dike.

Guide banks are usually earthen embankment faced with rock slope protection. Optimum shape and length of guide dikes will be different for each site. Field experience has shown that an elliptical shape with a major to minor axis ratio of 2.5:1 is effective in reducing turbulence. The length is dependant on the ratio of flow diverted from the flood plain to flow in the first 100 feet of waterway under the bridge. If the use of another shape dike, such as a straight dike, is required for practical reasons more scour should be expected at the upstream end of the dike. The bridge end will generally not be immediately threatened should a failure occur at the upstream end of a guide dike.

Toe dikes are sometimes needed downstream of the bridge end to guide flow away from the structure so that redistribution in the flood plain will not cause erosion damage to the embankment due to eddy currents. The shape of toe dikes is of less importance than it is with upstream guide banks.

For further information on spur dike and guide bank design procedures, refer to Section 6.4 of HDS No. 6. General design considerations and guidance for evaluating scour and stream stability at highway bridges is contained in HEC-18, HEC-20, and HEC-23.

CHAPTER 880
CURRENTLY NOT IN USE